CHAPTER 3 – OPEN CHANNELS

3.1 GENERAL

Open channels are a natural or constructed conveyance for water in which the water surface is exposed to the atmosphere, and the gravity force component in the direction of motion is the driving force. The principles of open channel flow hydraulics are applicable to all drainage facilities including culverts and storm drains.

The two basic types of open channels encountered in the City of Fort Smith are natural channels and drainage ditches.

Natural channels are:

- undisturbed channels with their size and shape determined by natural forces,
- usually compound in cross section with a main channel for conveying low flows and a floodplain to transport flood flows, and
- usually shaped in cross section and plan form by the long-term history of sediment load and water discharge that they experience.

Drainage ditches are artificial channels which are:

- constructed channels with regular geometric cross sections, and
- unlined or lined with artificial or natural material to protect against erosion.

Although the principles of open-channel flow are the same regardless of the channel type, stream channels and drainage ditches will be treated separately in this chapter as needed.

3.2 DESIGN CRITERIA

Open channels shall be designed according to the criteria listed below.

3.2.1 *Natural Channels*

The following criteria apply to natural channels and may be revised as approved by the Engineering Department:

- If approved by the Engineering Department, natural channels (unaltered, with existing trees and vegetation, not relocated or channelized) may be used in new developments providing the channel will carry the design storm runoff without erosion problems and sufficient land is included in a drainage easement. No clearing shall be allowed within the drainage easement. Natural channels may only be used where all the criteria for natural channels can be met. Drainage ditches or underground enclosed storm sewers must be used where criteria cannot be met.

- Natural channels must have a minimum freeboard of 1 ft for the 25-yr storm event.

- 10-year flows greater than 50 cfs may be carried in natural channels. 10-year flows less than 50 cfs must be contained in an underground enclosed storm sewer.

- The hydraulic effects of drainageway encroachments shall be evaluated over a full range of frequency-based peak discharges from the 10-yr through 100-yr recurrence intervals in areas where houses or other structures are present.

- Where encroachments are necessary (such as street crossings, utility crossings, storm sewer outfalls, etc.), streambank stabilization shall be provided and shall include both upstream and downstream banks and the local site.

- For natural channels less than 500 feet in length, and not located within a regulatory floodplain, single-section analysis (Manning's Equation) may be used for design, provided that the channel does not discharge to a culvert, storm drain, or other obstruction. All other channels must be designed using step-backwater analysis.

- The maximum water surface elevation for the 100-year storm event shall be 1 foot lower than the floor elevation of adjacent buildings or structures.

3.2.2 *Drainage Ditches*

The following criteria apply to drainage ditches and may be revised as approved by the Engineering Department:

- Drainage ditches may only be used in locations where all criteria for drainage ditches can be met. Underground enclosed storm sewer must be used where criteria cannot be met.

- 10-year flows greater than 50 cfs may be carried in drainage ditches. 10-year flows less than 50 cfs must be contained in an underground enclosed storm sewer, with the following exception: 10-year flows less than 50 cfs may be carried in Open Channel Systems designed to treat the required Water Quality Volume, per Chapter 5 – Post Construction Stormwater Management.

- For roadside drainage ditches, a minimum 10 foot wide clear zone must be provided between the back of curb/edge of pavement and the top edge of side slope or channel wall.

- Drainage ditches used for side lot drainage shall be concrete-lined. Concrete-lined drainage ditches must extend to the rear of the lot.

- Drainage ditches must have a trapezoidal cross section with a bottom width that is equal to or exceeds 3 times the depth of flow for the design storm. The minimum bottom width shall be 4 feet.

- Side slopes for concrete-lined drainage ditches shall be 1V:2H or flatter. Side slopes for earthen drainage ditches shall be 1V:4H or flatter.

- For concrete-lined drainage ditches, the top edge of concrete lining shall extend to the original ground level or to a point where an earthen slope can be constructed on a grade of 1V:4H or flatter. The design flow, plus freeboard, must be contained within the concrete-lined drainage ditch.

- Earthen drainage ditches may be used where the velocities from a 25-yr storm are less than 6 ft/s. All earthen drainage ditches shall be seeded or sodded immediately after their construction, and adequate measures shall be taken to prevent erosion. Special protections (such as headwalls, rip rap, grouted rip rap, gabions, etc.) will be required in all locations (such as bends, junctions, inlets or outlets of storm sewers, etc.) where erosion is likely.

- The minimum longitudinal grade for concrete-lined drainage ditches shall be 0.30%. The minimum longitudinal grade for earthen drainage ditches shall be 0.50%.

- Vertical wall concrete channels may be used in lieu of concrete drainage ditches. The minimum bottom width shall be 4 feet. Vertical wall concrete channels greater than 2 feet in depth must be fenced in on all sides. The fence shall be a minimum of 4 feet high and shall be chain link or other approved type. The fence shall have, as a minimum, 10 foot wide gates accessible by easement, placed no more than 400 feet apart. Where fence is required, a one foot wide, 6" thick, concrete lip shall be constructed adjacent to the tops of the vertical walls. Fence posts shall be set along the centerline of the one foot wide concrete lip.

- All drainage ditches and channels shall be located in street right-of-way or a public drainage easement.

- All drainage ditches and channels shall have a minimum freeboard of 1 ft.

- For drainage ditches and channels less than 500 feet in length, and not located within a regulatory floodplain, single-section analysis (Manning's Equation) may be used for design, provided that the ditch or channel does not discharge to a culvert, storm drain, or other obstruction. All other ditches and channels must be designed using step-backwater analysis.

- The maximum water surface elevation for the 100-year storm event shall be 1 foot lower than the floor elevation of adjacent buildings or structures.

3.3 OPEN CHANNEL FLOW

Design analysis of both natural and artificial channels proceeds according to the basic principles of open-channel flow (see References *(3)*, *(7)*). The basic principles of fluid mechanics—continuity, momentum and energy—can be applied to open-channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principal (or primary) objectives of open-channel flow analysis. The following equations are the most commonly used to analyze open channel flow:

3.3.1 *Continuity Equation*

The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of one-dimensional, steady flow of an incompressible fluid, it assumes the simple form:

$$Q = A_1 V_1 = A_2 V_2 \tag{3.1}$$

where:

Q = discharge, ft^3/s
A = cross-sectional area of flow, ft^2
V = mean cross-sectional velocity, ft/s (which is perpendicular to the cross section)

The subscripts 1 and 2 refer to successive cross sections along the flow path.

3.3.2 *Manning's Equation*

For a given depth of flow in an open channel with a steady, uniform flow, the mean velocity, V, can be computed with Manning's equation:

$$V = (1.486/n) R^{2/3} S^{1/2} \tag{3.2}$$

where:

V = velocity, ft/s
n = Manning's roughness coefficient
R = hydraulic radius = A/P, ft
P = wetted perimeter, ft
S = slope of the energy gradeline, ft/ft (Note: For steady uniform flow, S = channel slope, ft/ft)

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values. The selection of Manning's n is discussed in Section 3.4.2. The range of n values for various types of channels and floodplains is given in Table 3-1.

The continuity equation can be combined with Manning's equation to obtain the steady, uniform flow discharge as:

$$Q = (1.486/n)AR^{2/3}S^{1/2} \tag{3.3}$$

The conveyance represents the carrying capacity of a stream cross section based upon its geometry and roughness characteristics alone and is independent of the streambed slope.

The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross section and the flow distribution through the opening in a proposed stream crossing. It is also used to determine the velocity distribution coefficient, a (see Equation 3.4).

For a given channel geometry, slope and roughness and a specified value of discharge Q, a unique value of depth occurs in steady, uniform flow. It is called normal depth and is computed from Equation 3.3 by expressing the area and hydraulic radius in terms of depth. The resulting equation may require a trial-and-error solution.

TABLE 3-1. Values of Manning's Roughness Coefficient *n* (Uniform Flow)

Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
1. Earth, straight and uniform			
a. Clean, recently completed	0.016	0.018	0.020
b. Clean, after weathering	0.018	0.022	0.025
c. Gravel, uniform section, clean	0.022	0.025	0.030
d. With short grass, few weeds	0.022	0.027	0.033
2. Earth, winding and sluggish			
a. No vegetation	0.023	0.025	0.030
b. Grass, some weeds	0.025	0.030	0.033
c. Dense weeds or aquatic plans in deep channels	0.030	0.035	0.040
d. Earth bottom and rubble sides	0.025	0.030	0.035
e. Stony bottom and weedy sides	0.025	0.035	0.045
f. Cobble bottom and clean sides	0.030	0.040	0.050
3. Dragline-excavated or dredged			
a. No vegetation	0.025	0.028	0.033
b. Light brush on banks	0.035	0.050	0.060

4. Rock cuts			
a. Smooth and uniform	0.025	0.035	0.040
b. Jagged and irregular	0.035	0.040	0.050
5. Channels not maintained, weeds, and brush uncut			
a. Dense weeds, high as flow depth	0.050	0.080	0.120
b. Clean bottom, brush on sides	0.040	0.050	0.080
c. Same, highest stage of flow	0.045	0.070	0.110
d. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
1. Minor streams (top width at flood stage <100 ft)			
a. Streams on Plain			
1) Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2) Same as above, but more stones/weeds	0.030	0.035	0.040
3) Clean, winding, some pools/shoals	0.033	0.040	0.045
4) Same as above, but some weeds/stones	0.035	0.045	0.050
5) Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6) Same as 4, but more stones	0.045	0.050	0.060
7) Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8) Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1) Bottom: gravels, cobbles and few boulders	0.030	0.040	0.050
2) Bottom: cobbles with large boulders	0.040	0.050	0.070
2. Floodplains			
a. Pasture, no brush			
1) Short grass	0.025	0.030	0.035
2) High grass	0.030	0.035	0.050
b. Cultivated area			
1) No crop	0.020	0.030	0.040
2) Mature row crops	0.025	0.035	0.045
3) Mature field crops	0.030	0.040	0.050
c. Brush			
1) Scattered brush, heavy weeds	0.035	0.050	0.070
2) Light brush and trees, in winter	0.035	0.050	0.060
3) Light brush and trees, in summer	0.040	0.050	0.080
4) Medium to dense brush, in winter	0.045	0.070	0.110
5) Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1) Dense willows, summer, straight	0.110	0.150	0.200
2) Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3) Same as above, but with heavy growth of sprouts	0.050	0.060	0.080

4) Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5) Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3. Major Streams (top width at flood stage >100 ft)			
a. Regular section with no boulders or brush	0.025	—	0.060
b. Irregular and rough section	0.035	—	0.100

Source: Reference *(3)*.

If the normal depth is greater than critical depth, the slope is classified as a mild slope while, on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

3.3.3 *Velocity Distribution Coefficient*

The flow velocity may not be uniform in a channel cross section due to the presence of free surface, friction along the channel boundary, and change in alignment and cross section. As a result of nonuniform distribution of velocities in a channel section, the velocity head of an open channel is usually greater than the average velocity head computed as $(Q/A_t)^2/2g$. A weighted average value of the velocity head is obtained by multiplying the average velocity head, above, by a velocity distribution coefficient, α, defined as:

$$\alpha = \frac{\sum_{i=1}^{n}(K_i^3 / A_i^2)}{(K_t^3 / A_t^2)} \tag{3.4}$$

where:

K_i = conveyance in subsection (see Equation 3.5), ft^3/s
K_t = total conveyance in section (see Equation 3.5), ft^3/s
A_i = cross-sectional area of subsection, ft^2
A_t = total cross-sectional area of section, ft^2
n = number of subsections

3.3.4 *Conveyance*

In channel analysis, it is often convenient to group the channel cross section properties in a single term called the channel conveyance K:

$$K = (1.486/n)AR^{2/3} \tag{3.5}$$

and then Equation 3.3 can be written as:

$$Q = KS^{1/2} \tag{3.6}$$

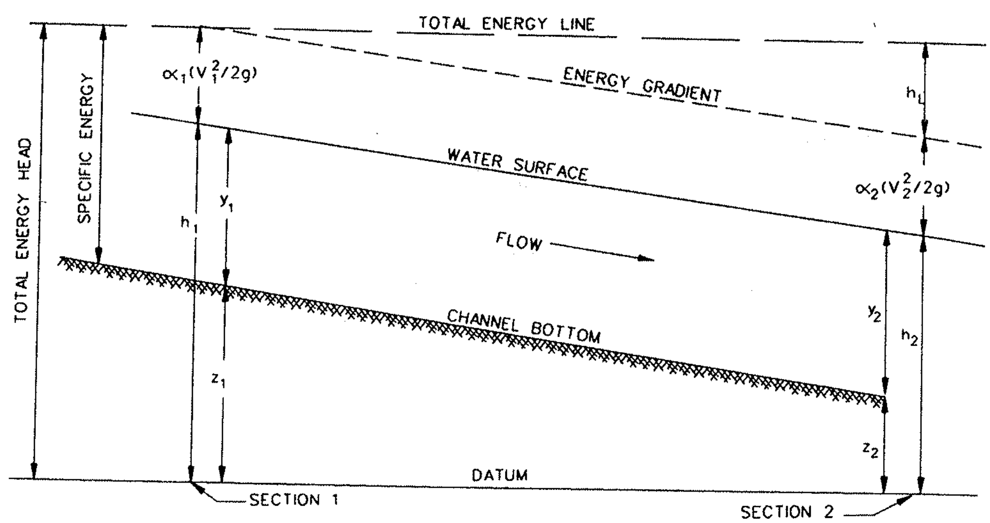
3.3.5 *Energy Equation*

The energy equation expresses conservation of energy in open channel flow expressed as energy per unit weight of fluid, which has dimensions of length and is therefore called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). These energy heads are scalar quantities that give the total energy head at any cross section when added. Written between an upstream open channel cross section designated 1 and a downstream cross section designated 2 (see Figure 3-1), the energy equation is:

$$h_1 + \alpha_1 (V_1^2/2g) = h_2 + \alpha_2 (V_2^2/2g) + h_L \tag{3.7}$$

where:

h_1, h_2 = the upstream and downstream stages, respectively, ft
α_1, α_2 = the upstream and downstream velocity distribution coefficients, respectively
V = mean velocity, ft/s
h_L = head loss due to local cross-sectional changes (minor loss) and boundary resistance, ft



Source: Reference *(5)*.

FIGURE 3-1. Terms in the Energy Equation

The stage, h, is the sum of the elevation head, z, at the channel bottom and the pressure head or depth of flow, y; i.e., $h = z + y$. The terms in the energy equation are illustrated graphically in Figure 3-1. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected.

3.4 HYDRAULIC ANALYSIS

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness and slope. The depth and velocity of flow are necessary for the design or analysis of roadway drainage structures.

The step-backwater method shall be used to compute the complete water surface profile in a stream reach or drainage ditch. Occasionally, the designer may need to use a more detailed method of analysis than the computation of a water surface profile using the step-backwater method. Special analysis techniques include two-dimensional analysis, water and sediment routing, and unsteady flow analysis.

3.4.1 *Cross Sections*

Cross-sectional geometry of streams is defined by coordinates of lateral distance and ground elevation that locate individual ground points. The cross section is taken normal to the flow direction along a single, straight line where possible but, in wide floodplains or bends, it may be necessary to use a section along intersecting straight lines; i.e., a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors.

Cross sections should be located to be representative of the subreaches between them. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross sections taken at shorter intervals to better model the change in conveyance.

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as for overbank flows. The conveyances of each subsection are computed separately to determine the flow distribution and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection (Reference *(4)*). Selection of cross sections and the vertical subdivision of a cross section are shown in Figure 3-2.

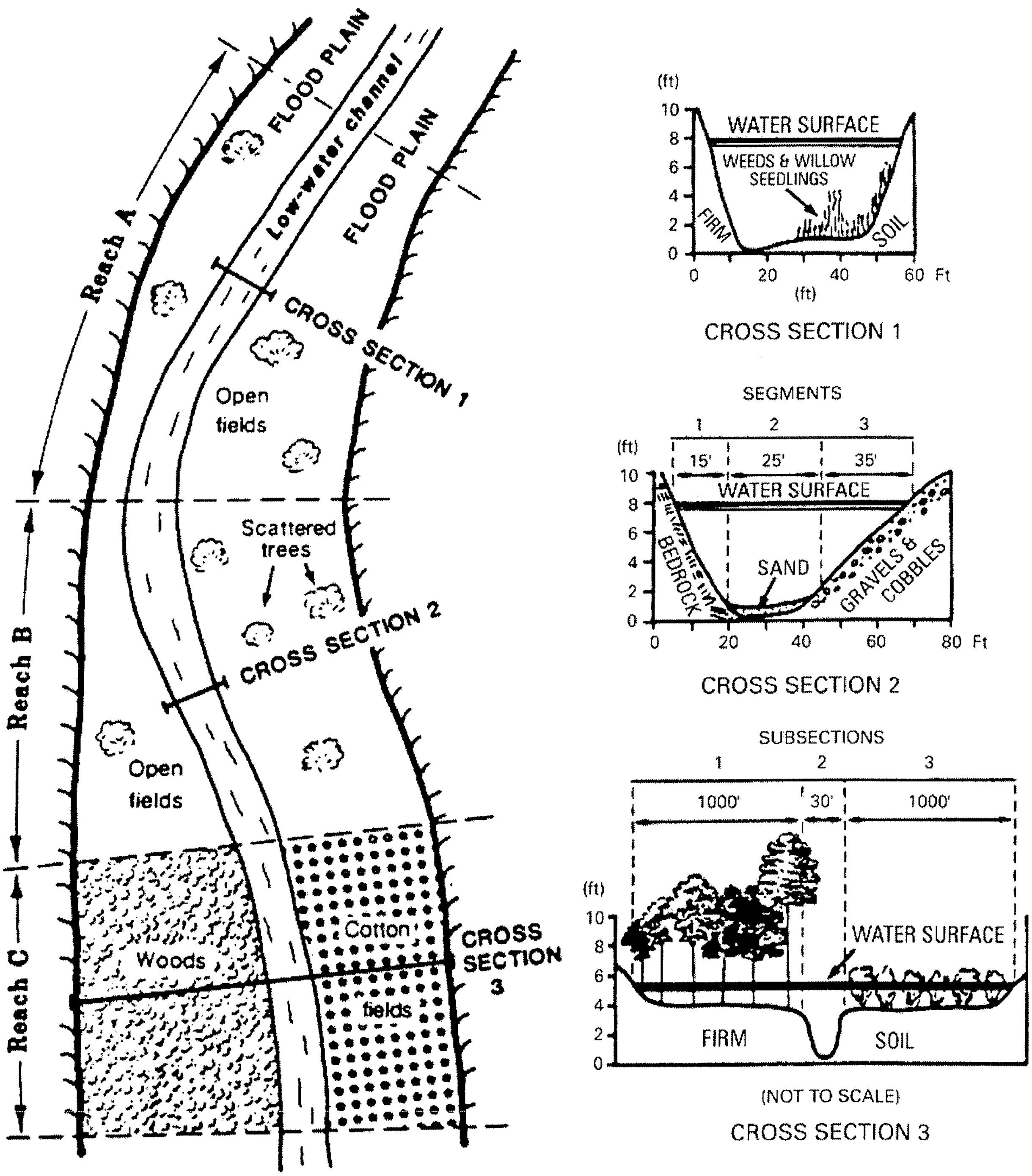


FIGURE 3-2. Hypothetical Cross Section Showing Reaches, Segments and Subsections Used in Assigning *n* Values

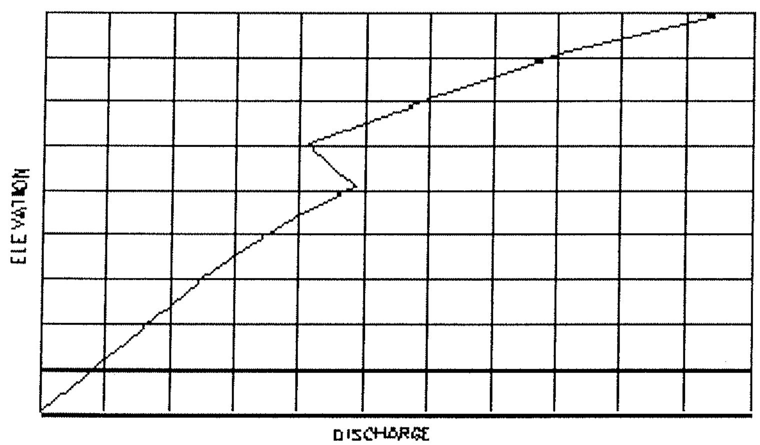
Source: Reference *(1)*.

3.4.2 *Manning's n Value Selection*

Manning's n is affected by many factors and its selection in natural channels depends heavily on engineering experience. Pictures of channels and floodplains for which the discharge has been measured and Manning's n has been calculated are very useful (see References *(1)*, *(2)*). For situations lying outside the engineer's experience, a more regimented approach is presented in Reference *(1)*. Once the Manning's n values have been selected, it is highly recommended that they be verified or calibrated with historical high-water marks and/or gaged streamflow data.

3.4.3 *Switchback Phenomenon*

If the cross section is improperly subdivided, the mathematics of Manning's equation causes a switchback. A switchback results where the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area that causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed that is lower than the discharge based upon the lower water depth. More subdivisions within such cross sections should be used to avoid the switchback.



Switchback Phenomenon

This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross section being used in a step-backwater program. For this reason, the cross section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n-value itself may be the same in adjacent subsections.

3.4.4 *Single-Section Analysis*

The single-section analysis method (slope-area method) is simply a solution of Manning's equation for the normal depth of flow given the discharge and cross section properties including geometry, slope, and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either artificial or natural stream channels. Nevertheless, the single-section method is often used to design artificial channels for uniform flow as a first approximation and to develop a stage-discharge rating curve in a stream channel for tailwater determination at a culvert or storm drain outfall.

3.4.5 *Step-Backwater Analysis*

Step-backwater analysis is useful for determining unrestricted water surface profiles where a roadway crossing is planned and for analyzing how far upstream the water surface elevations are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the FHWA/USGS program WSPRO *(6)* or USACE HEC-RAS *(9–11)* be used. Special analysis techniques should be considered for complex situations where a step-backwater analysis might not give the desired level of accuracy.

The WSPRO program has been designed to provide a water surface profile for six major types of open channel flow situations:

- unconstricted flow,
- single-opening bridge,
- bridge opening(s) with spur dikes,
- single-opening embankment overflow, and
- multiple alternatives for a single site and multiple openings.

The HEC-RAS program, developed by USACE, is widely used for calculating water surface profiles for steady, gradually varied flow in a natural or constructed channel. Both subcritical and supercritical flow profiles can be calculated. The effects of bridges, culverts, weirs and structures in the floodplain may be also considered in the computations. These programs are also designed for application in floodplain management and flood insurance studies.

3.4.5.1 Step-Backwater Methodology

The computation of water surface profiles by WSPRO and HEC-RAS is based on the standard-step method in which the stream reach of interest is divided into a number of subreaches by cross sections spaced such that the flow is gradually varied in each subreach. The energy equation is then solved in a step-wise fashion for the stage at one cross section based on the stage at the previous cross section.

The method requires definition of the geometry and roughness of each cross section as discussed in Section 3.4.1. Manning's n values can vary both horizontally across the section and vertically. Expansion and contraction head loss coefficients, variable main channel and overbank flow lengths, and the method of averaging the slope of the energy grade line can all be specified.

To amplify on the methodology, the energy equation is repeated from Section 3.3.5:

$$h_1 + \alpha_1 (V_1^2/2g) = h_2 + \alpha_2 (V_2^2/2g) + h_L \qquad (3.8)$$

where:

h_1, h_2 = the upstream and downstream stages, respectively, ft
α_1, α_2 = the upstream and downstream velocity distribution coefficients, respectively
V = mean velocity, ft/s
h_L = head loss due to local cross-sectional changes (minor loss) and boundary resistance, ft

The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow, y; i.e., $h = z + y$. The energy equation is solved between successive stream reaches with nearly uniform roughness, slope and cross-sectional properties.
The total head loss (h_L) is calculated from:

$$h_L = K_m \left|[(\alpha_1 V_1^2/2g) - (\alpha_2 V_2^2/2g)]\right| + \overline{S}_f L \qquad (3.9)$$

where:

K_m = expansion or contraction loss coefficient
$\overline{S}_f$ = the mean slope of the energy grade line evaluated from Manning's equation and a selected averaging technique, ft/ft
L = discharge-weighted or conveyance-weighted reach length, ft

These equations are solved numerically in a step-by-step procedure called the Standard-Step Method from one cross section to the next.

The loss coefficient K_m is used to calculate the expansion or contraction loss between cross sections. Typical values for K_m are 0.1 for a gradual contraction, 0.3 for a sudden contraction, 0.3 for a gradual expansion and 0.5 for a sudden expansion. The default values of the minor loss coefficient K_m are 0.0 and 0.1 for contractions and 0.5 and 0.3 for expansions in WSPRO and HEC-RAS, respectively. Refer to the HEC-RAS *Hydraulic Reference Manual (10)* for guidance on selecting expansion and contraction loss coefficients.

WSPRO calculates a conveyance-weighted reach length, L, as:

$$L = [(L_{lob}K_{lob} + L_{ch}K_{ch} + L_{rob}K_{rob})/(K_{lob} + K_{ch} + K_{rob})] \tag{3.10}$$

where:

L_{lob}, L_{ch}, L_{rob} = flow distance between cross sections in the left overbank, main channel and right overbank, respectively, ft

K_{lob}, K_{ch}, K_{rob}= conveyance in the left overbank, main channel and right overbank, respectively, of the cross section with the unknown water surface elevation

HEC-RAS calculates a discharge-weighted reach length, L, as:

$$L = [(L_{lob}\bar{Q}_{lob} + L_{ch}\bar{Q}_{ch} + L_{rob}\bar{Q}_{rob})/(\bar{Q}_{lob} + \bar{Q}_{ch} + \bar{Q}_{rob})] \tag{3.11}$$

where:

L_{lob}, L_{ch}, L_{rob} = flow distance between cross sections in the left overbank, main channel and right overbank, respectively, ft

$\bar{Q}_{lob}$, $\bar{Q}_{ch}$, $\bar{Q}_{rob}$ = arithmetic average of flows between cross section for the left overbank, main channel and right overbank, respectively, ft^3/s

WSPRO and HEC-RAS allow the user the following options for determining the friction slope, $\bar{S}_f$:

- Average conveyance equation:

$$\bar{S}_f = [(Q_u + Q_d)/(K_u + K_d)]^2 \tag{3.12}$$

- Average friction slope equation:

$$\bar{S}_f = (S_{fu} + S_{fd})/2 \tag{3.13}$$

- Geometric mean friction slope equation:

$$\bar{S}_f = (S_{fu}S_{fd})^{1/2} \tag{3.14}$$

- Harmonic mean friction slope equation:

$$\bar{S}_f = (2S_{fu}S_{fd})/(S_{fu} + S_{fd}) \tag{3.15}$$

where:

Q_u, Q_d = discharge at the upstream and downstream cross sections, respectively, ft^3/s

K_u, K_d = conveyance at the upstream and downstream cross sections, respectively, ft^3/s

S_{fu}, S_{fd} = friction slope at the upstream and downstream cross sections, respectively, ft/ft

The default option is the geometric mean friction slope equation in WSPRO and the average conveyance equation in HEC-RAS.

3.4.5.2 Profile Computation

Water surface profile computation requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section but, in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can either be found by the single-section method (slope-area method) or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the several profiles do not converge, then the stream reach may need to be extended downstream, or a shorter cross section interval should be used, or the range of starting water surface elevations should be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis (see Figure 3-3).

Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define the limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the structure site, and continued a sufficient distance upstream to accurately determine the impact of the structure on upstream water surface profiles (see Figure 3-4).

USACE *(8)* developed equations for determining upstream and downstream reach lengths as follows:

$$Ldn = 8000\ (HD^{0.8}/S) \tag{3.16}$$

$$Lu = 10{,}000\ [(HD^{0.6})(HL^{0.5})]/S \tag{3.17}$$

where:

Ldn = downstream study length (along main channel), ft (for normal depth starting conditions)
Lu = estimated upstream study length (along main channel), ft (required for convergence of the modified profile to within 0.1 ft of the base profile)
HD = average hydraulic depth (1% chance event flow area divided by the top width), ft
S = average reach slope, ft/mi
HL = headloss ranging between 0.5 ft and 5 ft at the channel crossing structure for the 1% chance flood, ft

References *(4)* and *(8)* are very valuable sources of additional guidance on the practical application of the step-backwater method to highway drainage problems involving open channels. These references contain more specific guidance on cross section determination, location and spacing and stream reach determination. Reference *(8)* investigates the accuracy and reliability of water surface profiles related to n-value determination and the survey or mapping technology used to determine the cross section coordinate geometry.

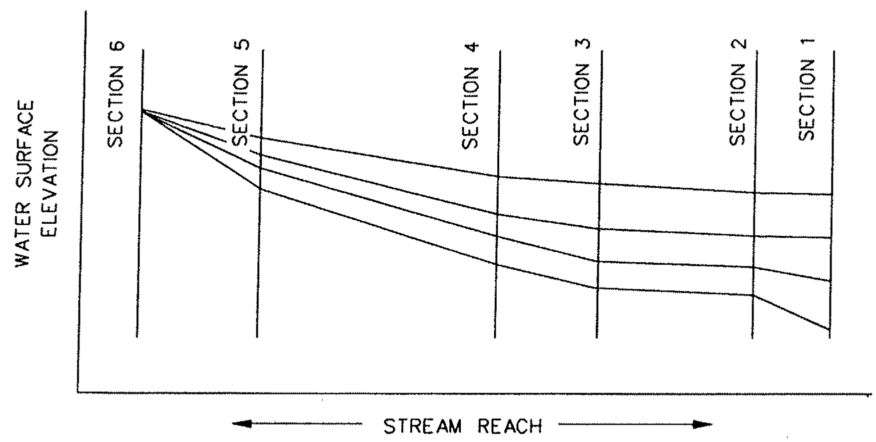


FIGURE 3-3. Profile Convergence Pattern Backwater Computation

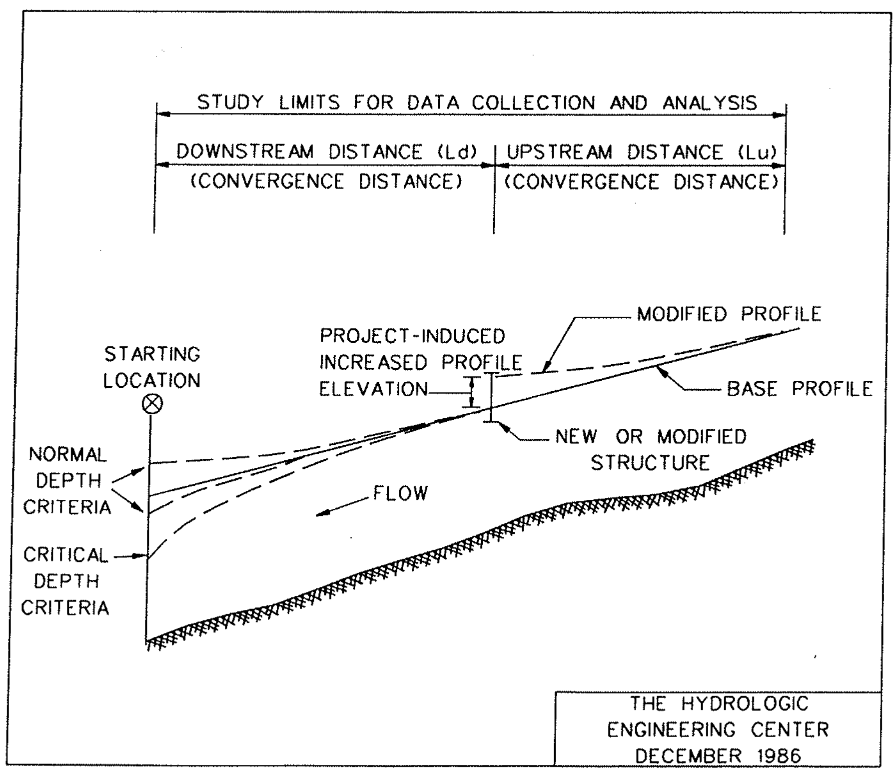


FIGURE 3-4. Profile Study Limits

Source: Reference *(8)*.

3.4.5.3 Computation Procedure

A sample procedure is taken from Reference *(12)*. An example problem using this procedure is provided in Appendix 3A.

A convenient form for use in calculating water surface profiles is shown in Figure 3-5. In summary, Columns 2 and 4 through 12 are devoted to solving Manning's equation to obtain the energy loss due to friction; Columns 13 and 14 contain calculations for the velocity distribution across the section; Columns 15 through 17 contain the average kinetic energy; Column 18 contains calculations for "other losses" (expansion and contraction losses due to interchanges between kinetic potential energies as the water flows); and Column 19 contains the computed change in water surface elevation. Conservation of energy is accounted for by proceeding from section to section down the computation form.

Column 1 CROSS SECTION NO., is the cross section identification number. Miles upstream from the mouth are recommended.

Column 2 ASSUMED, is the assumed water surface elevation that must agree with the resulting, computed water surface elevation within + 0.05 ft, or some allowable tolerance, for trial calculations to be successful.

Column 3 COMPUTED, is the rating curve value for the first section but, thereafter, is the value calculated by adding *WS* to the computed water surface elevation for the previous cross section.

Column 4 A, is the cross section area. If the section is complex and has been subdivided into several parts (e.g., left overbank, channel and right overbank), use one line of the form for each subsection and sum to get A_t, the total area of cross section.

Column 5 R, is the hydraulic radius. Use the same procedure as for Column 4 if section is complex, but do not sum subsection values.

Column 6 $R^{2/3}$, is 2/3 power of hydraulic radius.

Column 7 n, is Manning roughness coefficient.

Column 8 K, is conveyance and is defined as $(C_m A R^{2/3}/n)$ where C_m is 1.486. If the cross section is complex, sum subsection K values to get K_t.

Cross Section No.	Water Surface Elevation		Area	Hydraulic Radius (R)	$R^{2/3}$	n	K	$\overline{K_t}$	1000 $\bar{S}_f$	L	h_f	K^3/A^2	α	V	$\alpha V^2/2g$	$(\alpha V^2/2g)$	h_o	Water Surface Elevation
	Assumed	Computed																
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

FIGURE 3-5. Water Surface Profile Form

Column 9 $\bar{K}_t$, is average conveyance for the reach, and is calculated by $0.5(K_{td} + K_{tu})$ where subscripts D and U refer to downstream and upstream ends of the reach, respectively.

Column 10 $\bar{S}_f$, is the average slope through the reach determined by $(Q/\bar{K}_t)^2$.

Column 11 L, is the discharge-weighted or conveyance-weighted reach length.

Column 12 h_f, is energy loss due to friction through the reach and is calculated by $h_f = (Q/\bar{K}_t)^2 L = \bar{S}_f L$.

Column 13 $S(K^3/A^2)$, is part of the expression relating distributed flow velocity to an average value. If the section is complex, calculate one of these values for each subsection and sum all subsection values to get a total. If one subsection is used, Column 13 is not needed and Column 14 equals one.

Column 14 α, is the velocity distribution coefficient and is calculated by $S(K^3/A^2)/(K_t^3/A_t^2)$ where the numerator is the sum of values in Column 13 and the denominator is calculated from K_t and A_t.

Column 15 V, is the average velocity and is calculated by Q/A_t.

Column 16 $\alpha V^2/2g$, is the average velocity head corrected for flow distribution.

Column 17 $D(\alpha V^2/2g)$, is the difference between velocity heads at the downstream and upstream sections.

Column 18 h_o, is "other losses," and is calculated by multiplying either the expansion or contraction coefficient, K_m, times the absolute value of Column 17. A contraction occurs whenever the velocity head downstream is greater than the velocity head upstream. Likewise, an expansion occurs when the velocity head upstream is greater than the velocity head downstream.

Column 19 DWS, is the change in water surface elevation from the previous cross section. It is the algebraic sum of Columns 12, 17, and 18.

3.4.6 *Special Analysis Techniques*

Open channel flow problems sometimes arise that require a more detailed analysis than the computation of a water surface profile using the Standard-Step Method or the Direct-Step Method. More detailed analysis techniques include two-dimensional analysis, water and sediment routing and unsteady flow analysis. Several computer programs are available for these more detailed analysis techniques. HEC-RAS can be used to analyze one-dimensional, unsteady flow. Both RMA2 and FESWMS-2DH can be used to analyze two-dimensional, unsteady flow, and are also commonly used for 2-D modeling. The BRI-STARS model can be used to study complicated sedimentation problems.

3.5 REFERENCES

(1) Arcement, G. J., Jr., and V. R. Schneider. *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains.* FHWA-TS-84-204. FHWA, U.S. Department of Transportation, Washington, DC, 1984.

(2) Barnes, H. H., Jr. *Roughness Characteristics of Natural Channels.* U.S. Geological Survey Water Supply Paper 1849. U.S. Government Printing Office, Washington, DC, 1978.

(3) Chow, V. T. *Open Channel Hydraulics.* McGraw-Hill, New York, NY, 1959.

(4) Davidian, J. *Computation of Water Surface Profiles in Open Channels, Techniques of Water Resources Investigation.* Book 3, Chapter A15. U.S. Geological Survey, Washington, DC, 1984.

(5) Federal Highway Administration. *River Engineering for Highway Encroachments—Highways in the River Environment.* Hydraulic Design Series No. 6, FHWA-NHI-01-004. FHWA, U.S. Department of Transportation, Washington, DC, December 2001.

(6) Federal Highway Administration. *WSPRO User's Manual.* Version P60188, FHWA-SA-98-080. FHWA, U.S. Department of Transportation, Washington, DC, 1999.

(7) Henderson, F. M. *Open Channel Flow.* Macmillan, New York, 1966.*l of Hydraulic Engineering.* Vol. 11, No. 5, American Society of Civil Engineers, Reston, VA, May 1985.

(8) U.S. Army Corps of Engineers. *Accuracy of Computed Water Surface Profiles.* The Hydrologic Engineering Center, Davis, CA, December 1986.

(9) U.S. Army Corps of Engineers. *HEC-RAS, River Analysis System—Applications Guide.* Version 3.1. The Hydrologic Engineering Center, Davis, California, November 2002.

(10) U.S. Army Corps of Engineers. *HEC-RAS, River Analysis System, Hydraulic Reference Manual.* Version 3.1. The Hydrologic Engineering Center, Davis, CA, November 2002.

(11) U.S. Army Corps of Engineers. *HEC-RAS, River Analysis System, User's Manual.* Version 3.1. The Hydrologic Engineering Center, Davis, CA, November 2002.

(12) U.S. Army Corps of Engineers. Water Surface Profiles. *Hydrologic Engineering Methods for Water Resources Development.* Volume 6, HEC-IHD-0600. The Hydrologic Engineering Center, Davis, CA, July 1975.

APPENDIX 3A

EXAMPLE PROBLEM – STEP BACKWATER METHOD

The step-backwater procedure is illustrated in the following example.

Four cross sections along a reach are shown in Figures 3A-1 through 3A-4. Each cross section is separated by 500 ft and is subdivided according to geometry and roughness. The calculations shown in Figure 3A-5 represent one set of water-surface calculations. An explanation of Figure 3A-5 is in Section 3.4.5.3.

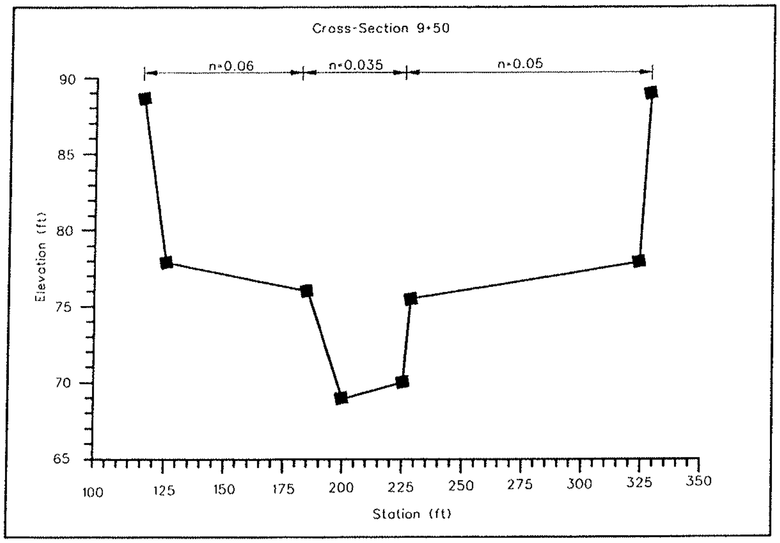


FIGURE 3A-1. Cross Section at Station 9+50 (farthest downstream)

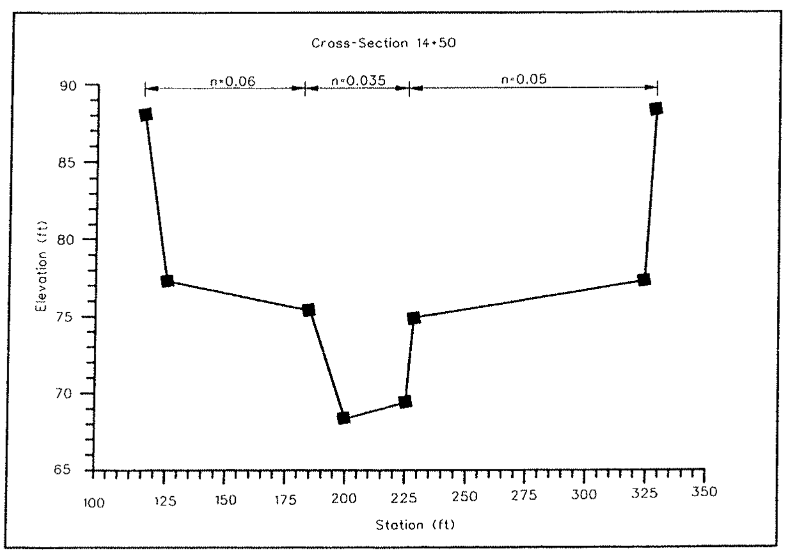


FIGURE 3A-2. Cross Section at Station 14+50

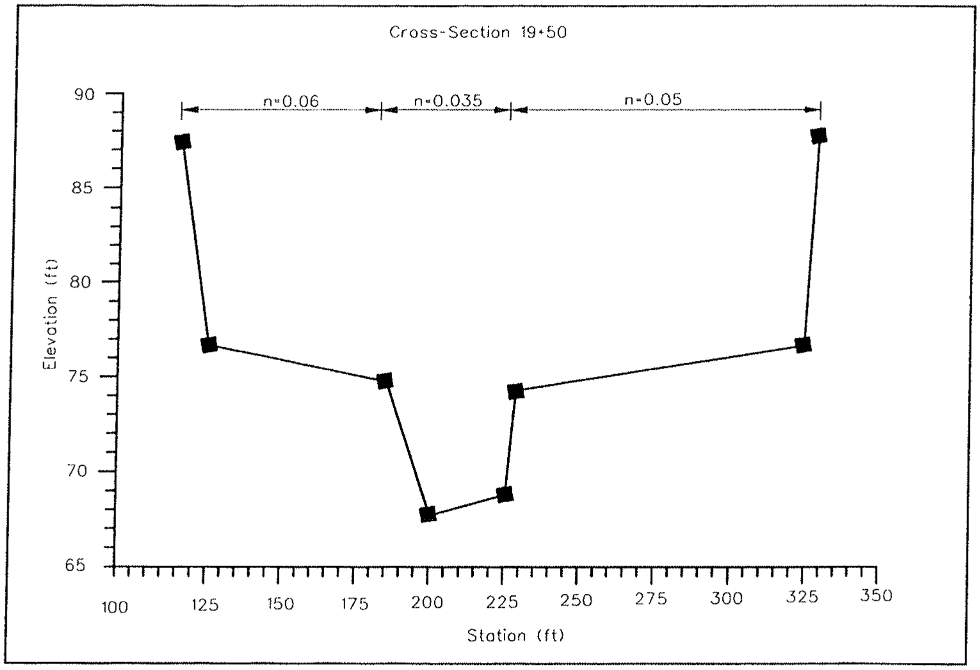


FIGURE 3A-3. Cross Section at Station 19+50

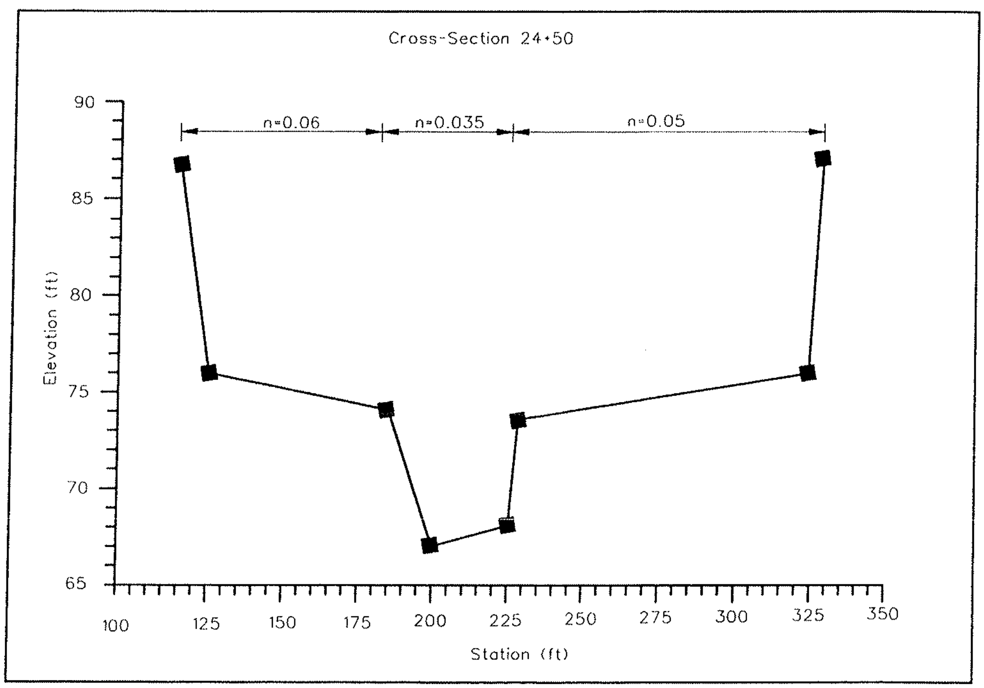


FIGURE 3A-4. Cross Section at Station 24+50 (farthest upstream)

FIGURE 3A-5. Example, Water Surface Profile Form

Cross Section No.	Water Surface Elevation		Area	R	$R^{2/3}$	n	K	$\overline{K}_t$	$1000S_f$	L	h_f	(K^3/A^2)	α	V	$\alpha V^2/2g$	$(\alpha V^2/2g)$	h_o	Water Surface Elevation
	Assumed	Computed																
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
9+50		79.30																
			191	2.23	1.71	0.060	8089					14,508,342						
			506	8.20	4.07	0.035	87437					2,610,866,774						
			146	1.28	1.18	0.050	5120					6,296,572						
			843				100646	100646	0.40	0	0	2,631,671,688	1.83	2.4	0.16	0.00	0.000	0.00
14+50	79.50	79.50																
			192	2.23	1.71	0.060	8131					14,582,414						
			507	8.20	4.07	0.035	87610					2,616,044,470						
			147	1.28	1.18	0.050	5155					6,339,446						
			846				100896	100771	0.40	500	0.20	2,636,966,330	1.84	2.4	0.16	0.00	0.000	0.20
19+50	80.22	79.71																
			227	2.66	1.92	0.060	10794					24,408,001						
			531	8.60	4.20	0.035	94688					3,010,888,601						
			196	1.71	1.43	0.050	8330					15,045,638						
			954				113812	107354	0.35	500	0.18	3,050,342,240	1.88	2.1	0.13	0.03	0.000	0.21
	79.66	79.87																
			182	2.17	1.68	0.060	7573					13,109,976						
			501	8.10	4.03	0.035	85722					2,509,603,778						
			136	1.18	1.12	0.050	4527					5,015,791						
			819				97822	99359	0.41	500	0.21	2,527,729,545	1.81	2.5	0.17	-0.04	0.000	0.17
24+50	79.89	79.30																
			130	1.87	1.52	0.060	4894					6,935,484						
			403	7.61	3.87	0.035	66217					1,787,681,873						
			84	0.95	0.97	0.050	2422					2,012,523						
			617				73532	93672	0.46	500	0	1,796,629,880	1.72	3.3	0.28	0.11	0.000	0.00

Q=2013 ft^3/s

CHAPTER 4 – CULVERTS

4.1 GENERAL

The function of a drainage culvert is to pass the design storm flow under a roadway or railroad without causing excessive backwater and without creating excessive downstream velocities. The designer shall keep energy losses and discharge velocities within reasonable limits when selecting a structure.

Culvert flow may be separated into two major types of flow, inlet or outlet control. Generally, the hydraulic control in a culvert will be at the culvert outlet if the culvert slope is less than the critical slope. Inlet control usually governs if the culvert slope is greater than critical slope. Other factors that determine how the culvert performs can be listed as physical makeup of the structure and the leading and trailing water surface profiles.

4.1.1 *Outlet Control*

For outlet control, factors such as type of opening, cross sectional area, barrel slope, barrel length, barrel roughness, and head losses due to tailwater are predominant in controlling the headwater of the culvert. These separately or conjointly create physical resistance that retards the flow of water. As the resistance accumulates, the flow begins to slow and increase in depth. At some point, when the resistance mounts, the water may cease to flow freely and back up in the structure and flood the upstream drainage basin.

4.1.2 *Inlet Control*

For inlet control, the entrance characteristics of the culvert are such that the entrance head losses are predominant in determining the headwater of the culvert. The barrel will carry water through the culvert faster than the water can enter the culvert.

4.2 DESIGN CRITERIA

- Design criteria for culverts shall include the following:

- Maximum headwater depth for the design storm shall be 1feet lower than the top of roadway or top of curb.

- Maximum headwater depth for the 100-year storm event shall be 1 foot lower than the floor elevation of adjacent buildings or structures.

- Minimum culvert diameter for round pipe shall be 15 inches. Minimum culvert size for arch pipe or elliptical pipe shall be 15 inch equivalent. Minimum size for box culverts shall be 4 feet by 4 feet.

- The minimum allowable fill or cover shall be 12 inches above the top of the culvert. For culverts under roadways there shall also be a minimum clearance of 6 inches from the top of the culvert to the bottom of the pavement base. Special box culverts designed to carry traffic on the top slab do not have to meet minimum allowable fill requirements.

- As a minimum, pipe culverts shall be constructed of Class III, Reinforced Concrete Pipe. Box Culverts shall also be constructed of reinforced concrete. All culverts shall be constructed with reinforced concrete headwalls/endwalls, slope walls, or flared-end sections at the inlet and outlet.

- Culvert length and slope shall be chosen to approximate existing topography.

- Culvert invert shall be aligned with the channel bottom and the skew angle of the stream.

- Tailwater depth may be calculated with Manning's Equation (Section 3.3.2) if Step Backwater Analysis is not required for the downstream channel. If the headwater elevation for a nearby downstream culvert or storm drain is greater than the normal depth for the channel, a Step Backwater Analysis shall be required.

- Energy dissipators will be required at culvert outlets in earthen channels when the discharge velocity exceeds 6 ft/s. Energy dissipators shall be designed in accordance with the *Hydraulic Design of Stilling Basins and Energy Dissipators (2)*, developed by the U.S. Bureau of Reclamation.

4.3 DESIGN PROCEDURE

The computations involved in selecting the smallest feasible culvert which can be used without exceeding the design headwater elevation can be summarized on the tabulation sheet from Reference *(1)*. The tabulation sheet is shown in Appendix 4A.

After the initial data has been entered into the tabulation sheet, the initial trial size must be entered in Column 1. The square feet of opening for the initial trial size may be estimated by dividing the design discharge by 10. After the initial trial size has been selected, the following procedure may be used to complete the culvert design:

4.3.1 *Step 1 – Perform Outlet Control Calculations*

Column 1: Enter the span and height dimensions (or diameter of pipe) of culvert.

Column 2: Enter the type of structure and design of entrance.

Column 3: Enter the design discharge or quotient of design discharge divided by the applicable denominator.

Column 4: Enter the Entrance Loss Coefficient, K_e. A list of Entrance Loss Coefficients from Reference *(3)* can be found in Appendix 4B, Outlet Control Data.

Column 5: Enter the head from the applicable outlet control nomograph. Nomographs from Reference *(3)* can be found in Appendix 4B, Outlet Control Data.

Column 6: Enter the critical depth from the appropriate nomograph. Nomographs from Reference *(3)* can be found in Appendix 4B, Outlet Control Data.

Column 7: For tailwater elevations less than the top of the culvert at the outlet, headwater is found by solving for h_o using the following equation:

$$h_o = (d_c + D)/2 \qquad (4.1)$$

where:
h_o = vertical distance in feet from culvert invert at outlet to the hydraulic grade line in feet
d_c = critical depth in feet
D = height of culvert opening in feet

Column 8: Enter the tailwater elevation from initial data shown at the top of the tabulation sheet.

Column 9: Enter the product of the culvert length times the slope.

Column 10: Headwater elevation required for culvert to pass flow in outlet control (HW_o) is computed by the following equation:

$$HW_o = H + h_o - LS \qquad (4.2)$$

Note: Use TW elevation in lieu of h_o where TW > h_o.

Additional trials may be required to determine the minimum barrel size. Space for additional trials is provided on the tabulation sheet.

4.3.2 *Step 2 – Perform Inlet Control Calculations*

Column 11: Enter ratio of headwater to height of structure from the applicable inlet control nomograph. Nomographs from Reference *(3)* can be found in Appendix 4C, Inlet Control Data.

Column 12: HW is derived by multiplying Column 11 by the height (or diameter) of culvert.

4.3.3 *Step 3 – Choose Controlling Headwater, Calculate Velocity*

Column 13: Enter the greater of the two headwaters (Column 10 or 12).

Column 14: If inlet control governs, outlet velocity equals Q/A, where A is defined by the cross sectional area of normal depth of flow in the culvert barrel. A hydraulic elements chart from Reference *(1)* has been included in Appendix 4D to assist in estimating normal depth of flow and velocity. Manning's Equation (3.3.2) may also be used.

If outlet control governs, outlet velocity equals Q/A, where A is the cross sectional area of flow in the culvert barrel at the outlet, based on the following:

- Critical depth if the tailwater is below critical depth.
- The tailwater depth if the tailwater is between critical depth and the top of the barrel.
- The height of the barrel if the tailwater is above the top of the barrel.

Columns 15 & 16: These columns are self-explanatory.

4.4 EXAMPLE PROBLEM

An example problem from Reference *(1)* is shown below:

Given:	Design Discharge	= 1,000 cfs
	Slope of Stream Bed	= 0.071 ft/ft
	Allowable Headwater Elevation	= 200.0
	Elevation of Outlet Invert	= 182.9
	Culvert Length	= 100 ft

Downstream channel approximates an 8 ft wide trapezoidal channel with 2:1 side slopes, Manning "n" of 0.03 and a slope of 0.071 feet per foot.

Required: Design a culvert that will adequately convey the design discharge. The culvert should have the smallest possible barrel to pass design Q without exceeding AHW elevation. Try a reinforced box culvert with "n" = 0.012.

Solution: The solution is shown on the tabulation sheet from Reference *(1)* in Appendix 4E, Example Problem.

4.5 REFERENCES

(1) Arkansas State Highway and Transportation Department. *Drainage Manual.* July 1982.

(2) Bureau of Reclamation. *Hydraulic Design of Stilling Basins and Energy Dissipators.* Engineering Monograph No. 25. U.S. Department of Interior, Bureau of Reclamation, Washington, DC. Third printing. 1974.

(3) Federal Highway Administration. *Hydraulic Design of Highway Culverts.* Hydraulic Design Series No. 5, FHWA-NHI-01-020. FHWA, U.S. Department of Transportation, Washington, DC, September 2001.

APPENDIX 4A

CULVERT TABULATION SHEET
(Source: AHTD)

FI RE 4A-1. Culvert Tabulation Sheet

FORM HYD 4-1

CULVERT COMPUTATIONS
(SQUARE AND BEVELED EDGES)

DESIGNER: ____________

PROJECT: ____________

DATE: ____________

HYDROLOGIC AND CHANNEL INFORMATION

HYDROLOGY

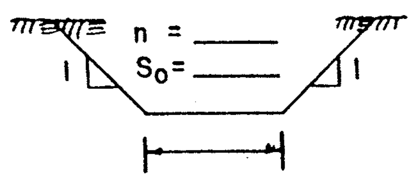
Q_1 _____ = _____ cfs

Q_2 _____ = _____ cfs

STREAM DATA

Tw_1 = _____

Tw_2 = _____



OUTLET CHANNEL
(APPROX. DIMENSIONS)

SKETCH

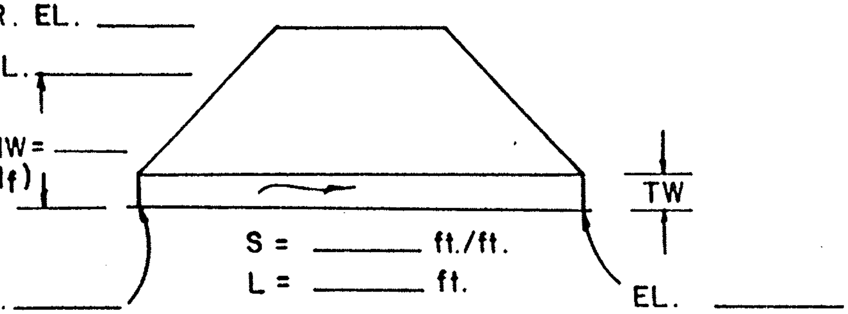
STATION: ____________

FIN. GR. EL. _____

HW_f EL. _____

AHW = _____
(H_f)

H_f EL. _____



TRIAL NO.	SIZE	STRUCTURE TYPE & ENTRANCE DESIGN	Q: Q/NB: $\frac{Q}{NBD^{3/2}}$	HEADWATER COMPUTATION: OUTLET CONTROL: (a) K_e	H	(b) d_c	(c) h_o	(d) TW	LS	(e) HW_o	INLET CONT.: $\frac{HW}{D}$ (f)	HW	CONTROL-LING HW	OUTLET VELOCITY ft./sec.	COST	COMMENTS
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16

(a) Entrance loss coefficient, Refer to Table 4-10, page 4-60.
(b) "d_c" cannot exceed D.
(c) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.
(d) TW = d_n in natural channel, or other downstream control.
(e) $HW_o = H + H_o - LS$
(f) Use Chart 4-7, page 4-64, for Conventional face.
Use Chart 4-8, page 4-65, for Beveled Edge.

APPENDIX 4B

OUTLET CONTROL DATA
(Source: FHWA)

Outlet Control, Full or Partly Full Entrance head loss

$$H_e = k_e \left(\frac{V^2}{2g} \right)$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be

TABLE 4B-1. Entrance Loss Coefficients

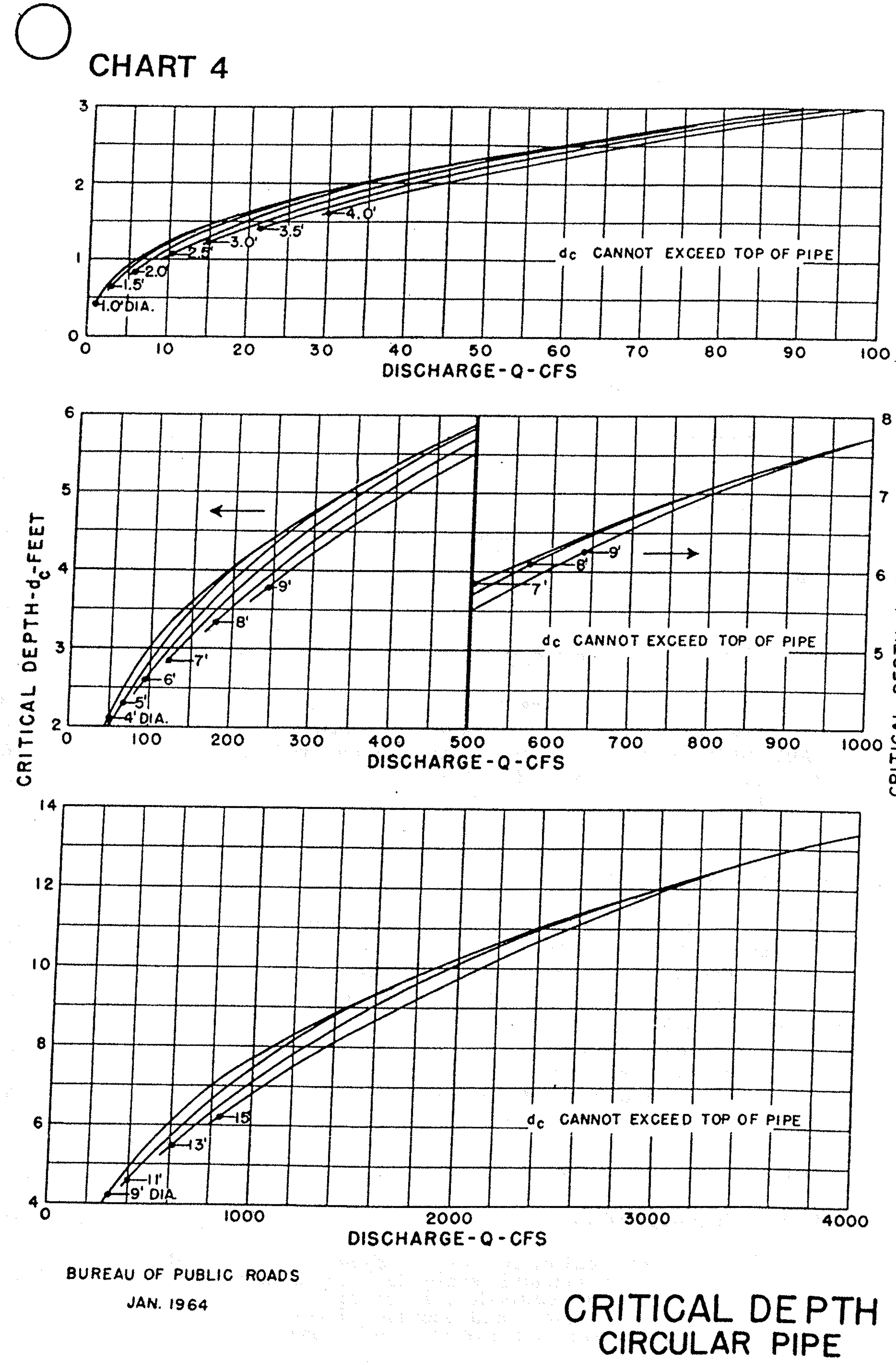


FIGURE 4B-1. Critical Depth, Circular Pipe

CHART 5

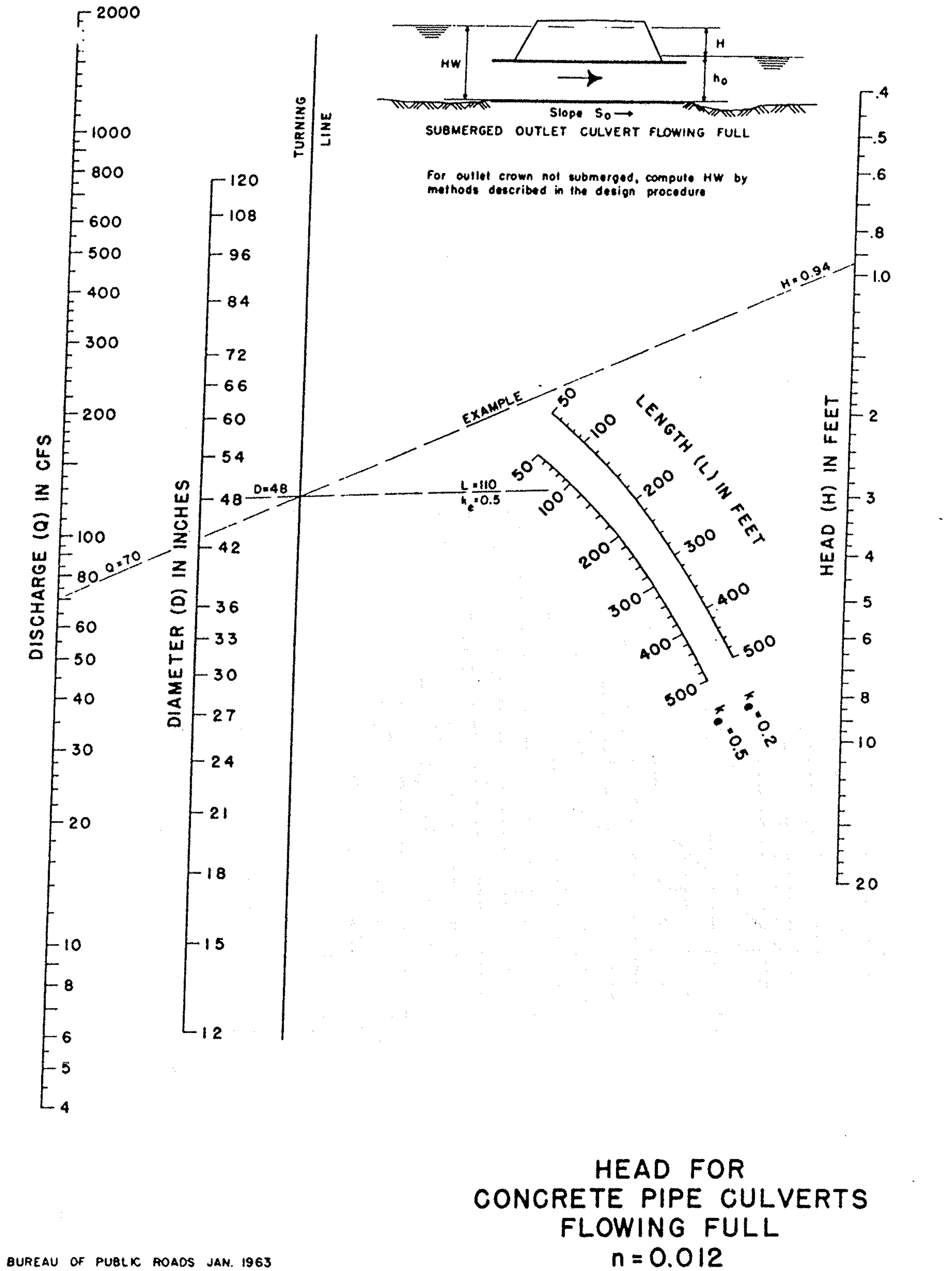


FIGURE 4B-2. Head for Concrete Pipe Culverts Flowing Full

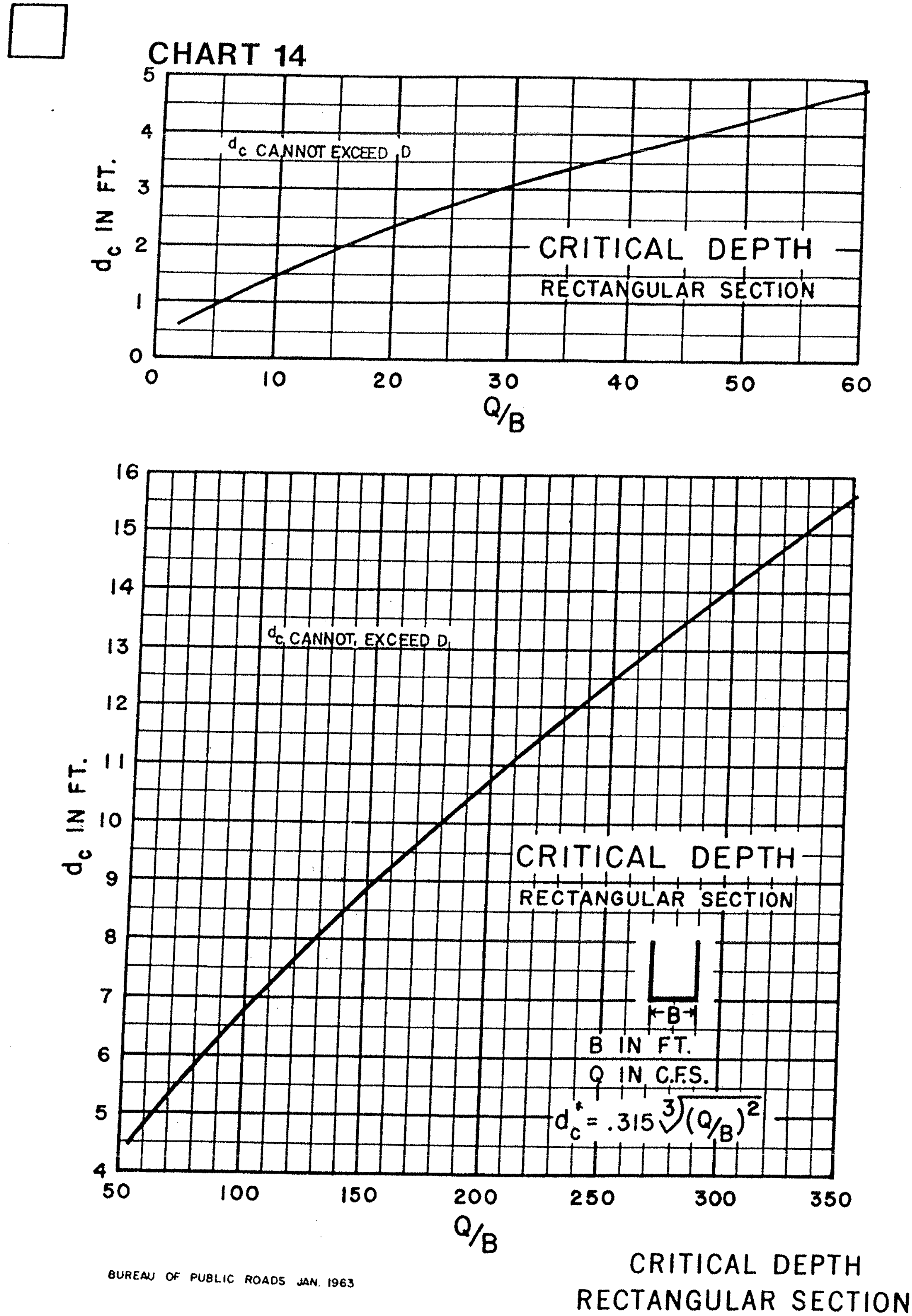
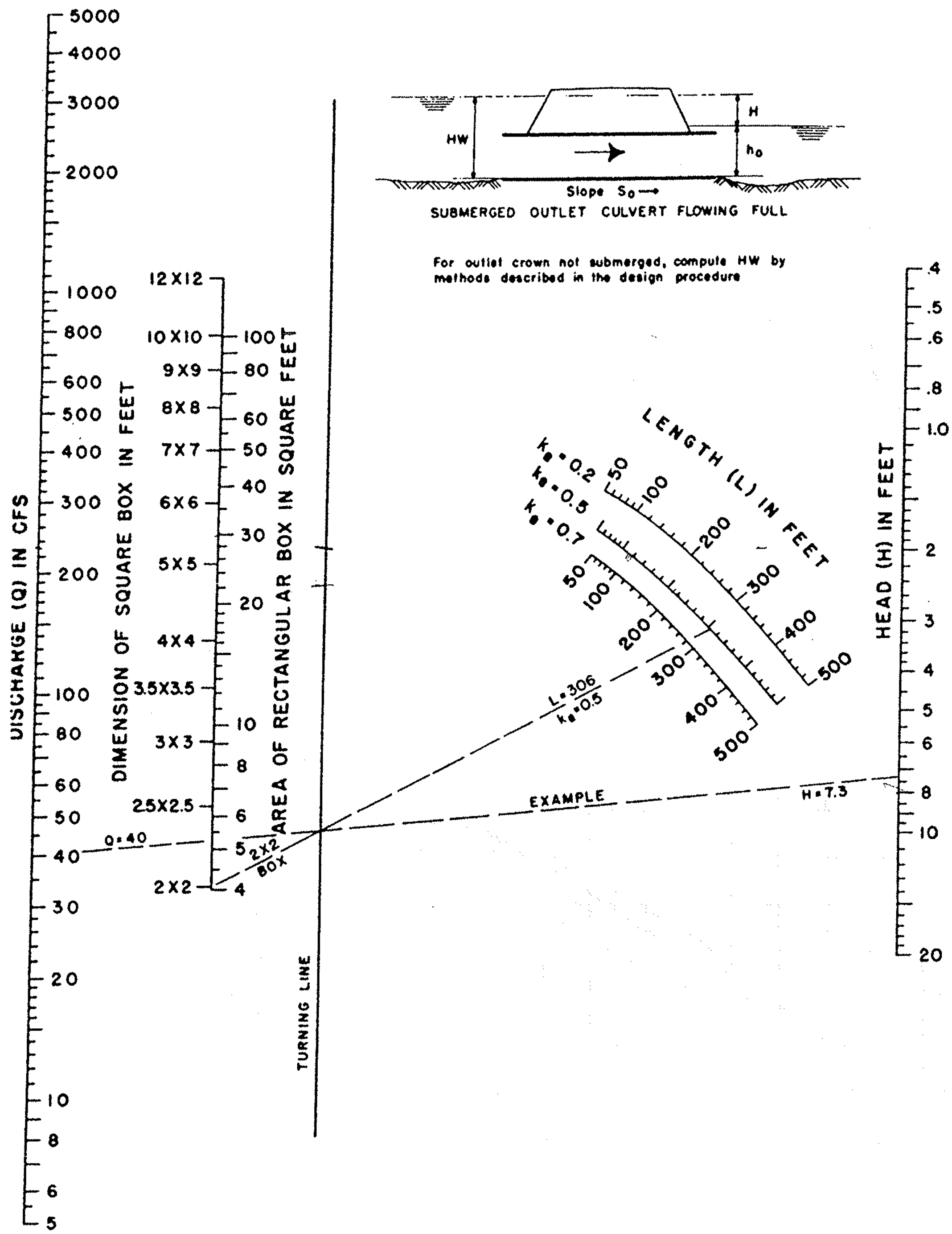


FIGURE 4B-3. Critical Depth, Rectangular Section



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

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FIGURE 4B-4. Head for Concrete Box Culverts Flowing Full

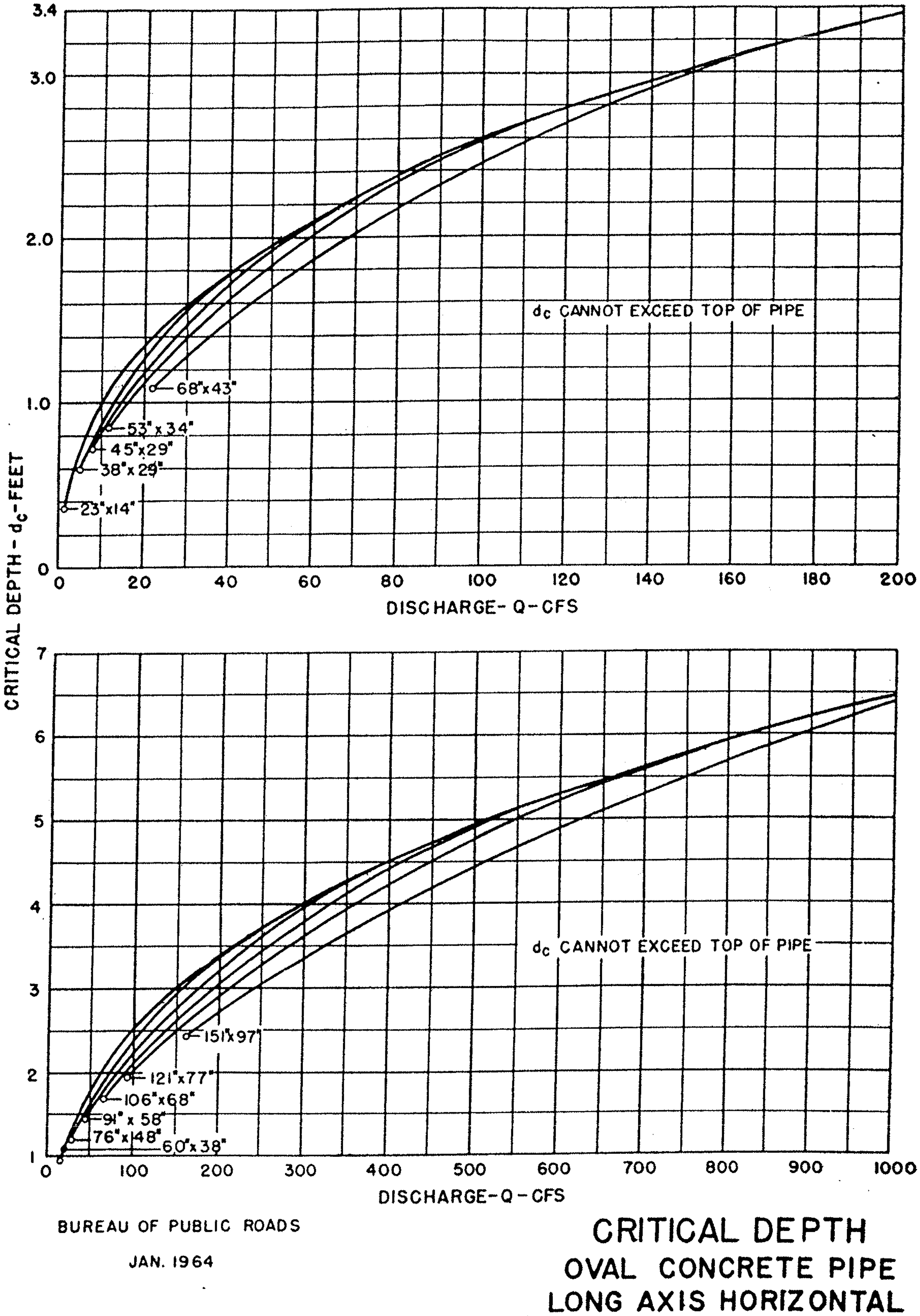
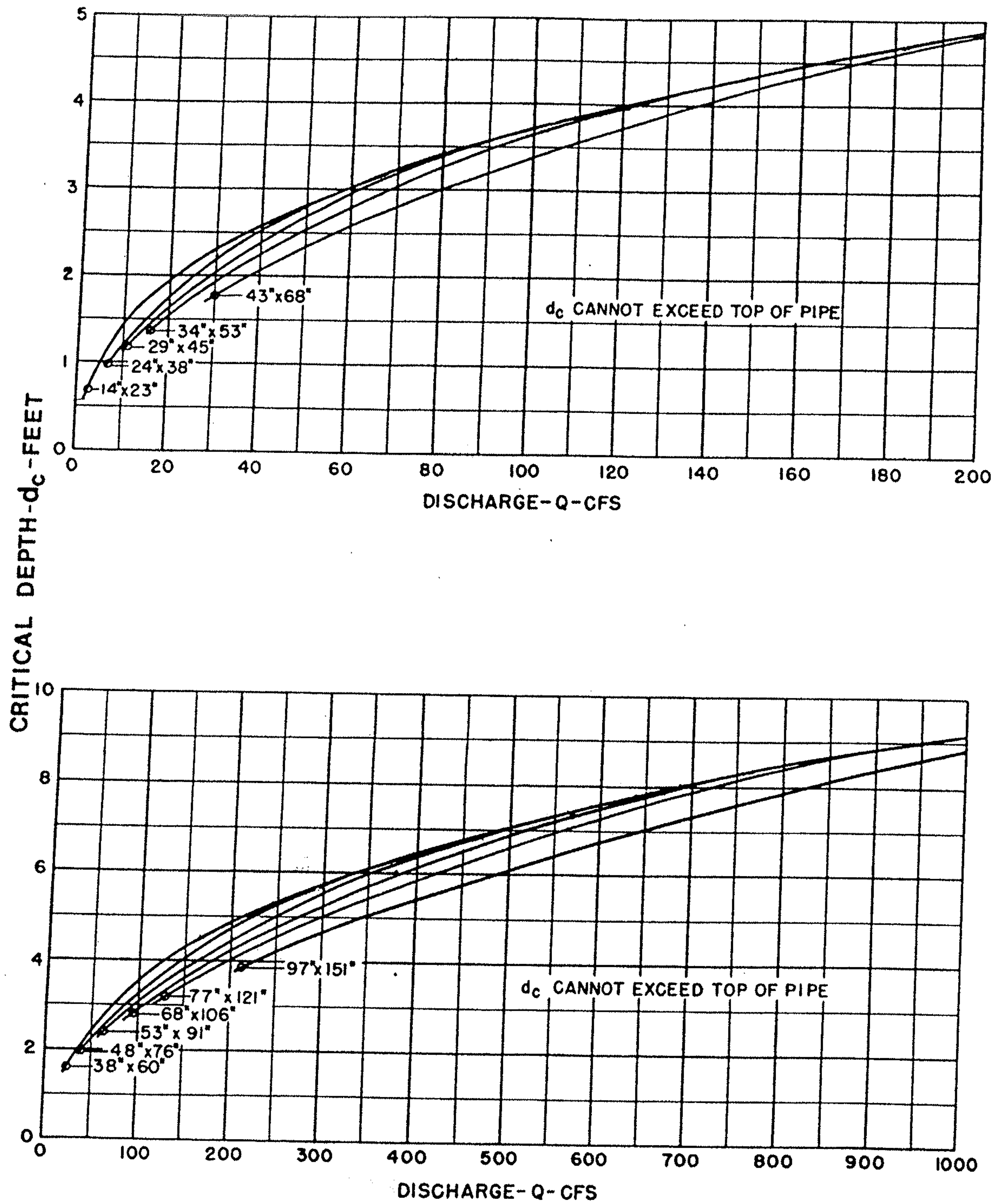


FIGURE 4B-5. Critical Depth, Oval Concrete Pipe, Long Axis Horizontal

CHART 32



BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL

FIGURE 4B-6. Critical Depth, Oval Concrete Pipe, Long Axis Vertical

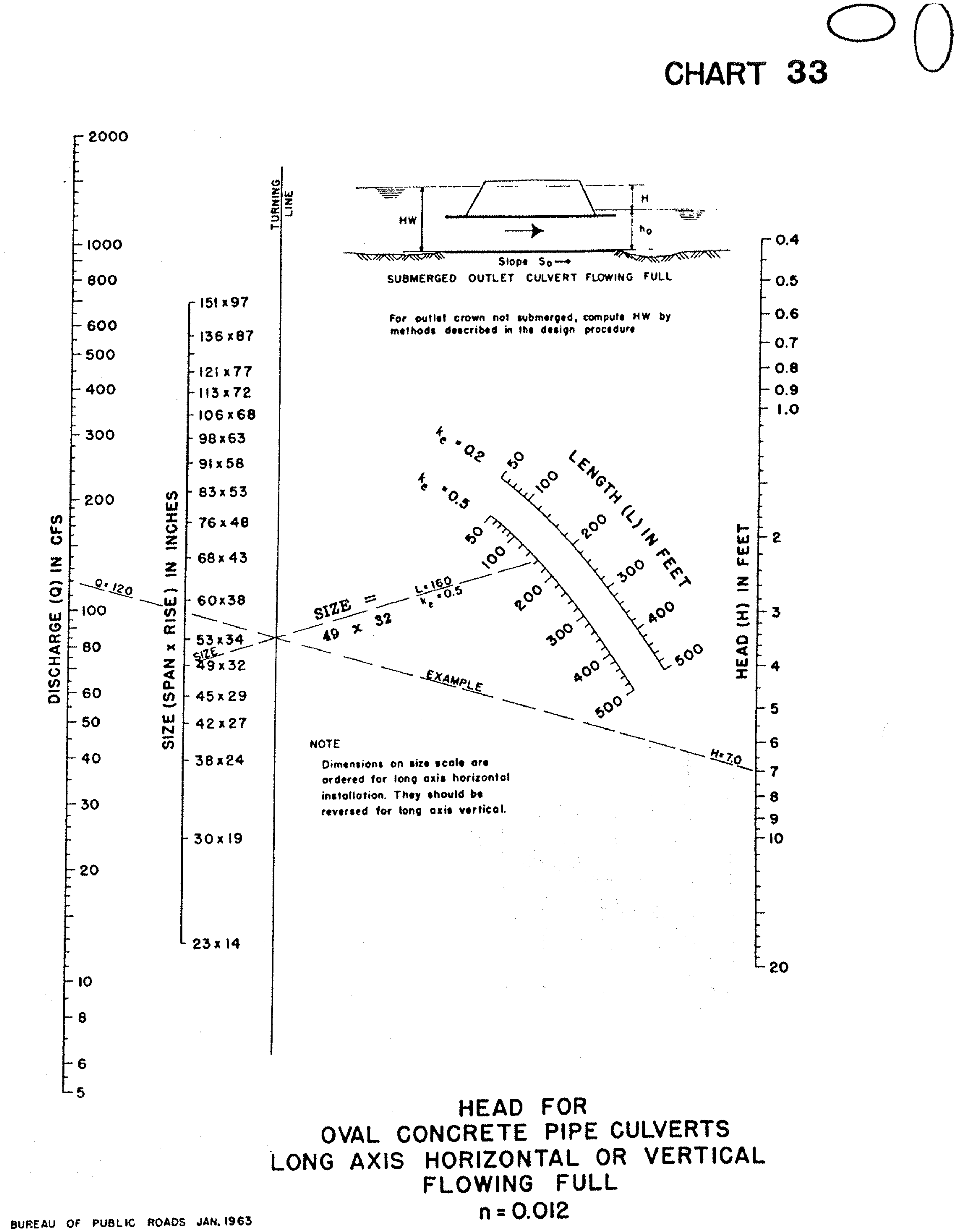


FIGURE 4B-7. Head for Oval Concrete Pipe Culverts, Long Axis Horizontal or Vertical Flowing Full

APPENDIX 4C

INLET CONTROL DATA
(Source: FHWA)

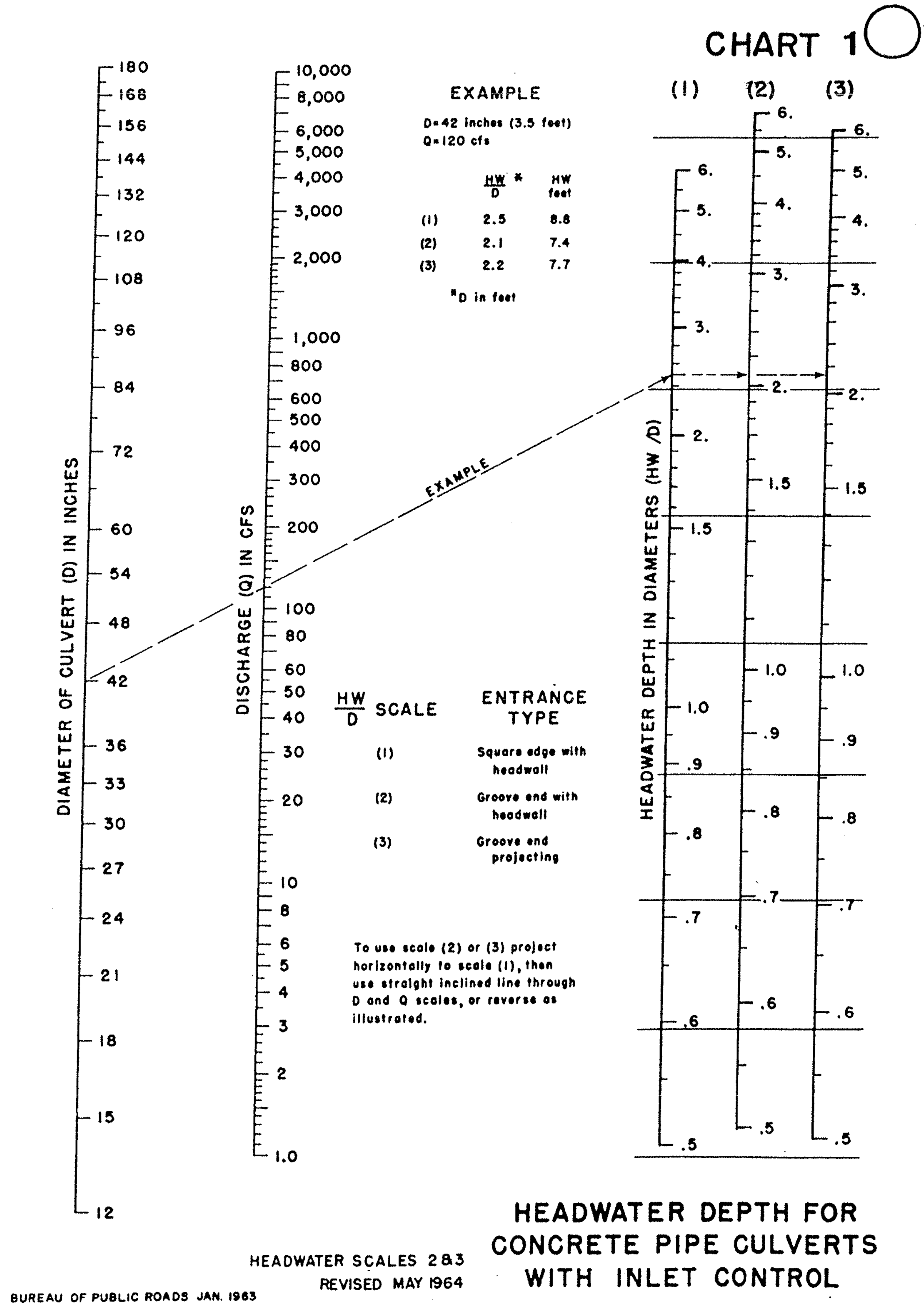


FIGURE 4C-1. Headwater Depth for Concrete Pipe Culverts With Inlet Control

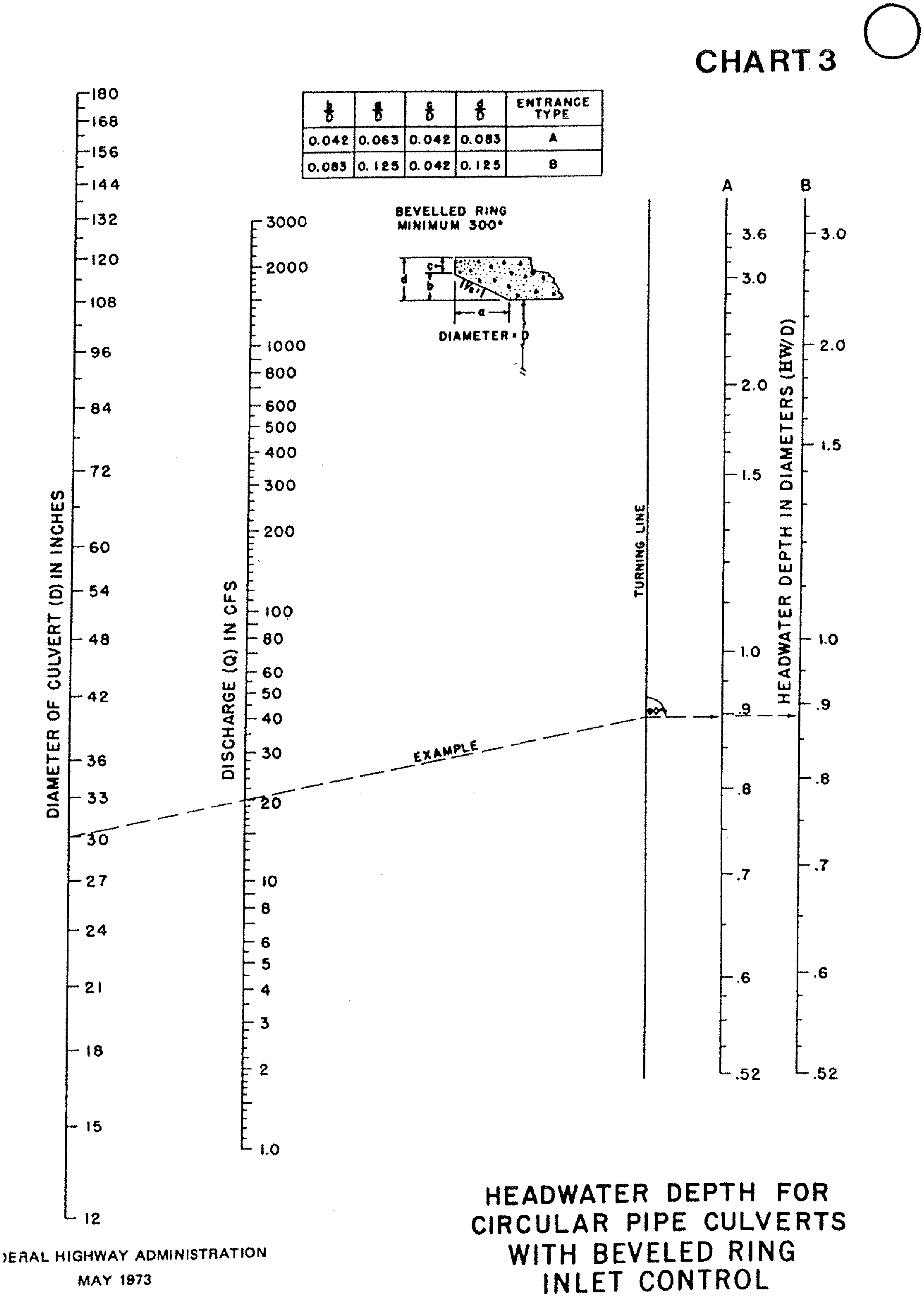
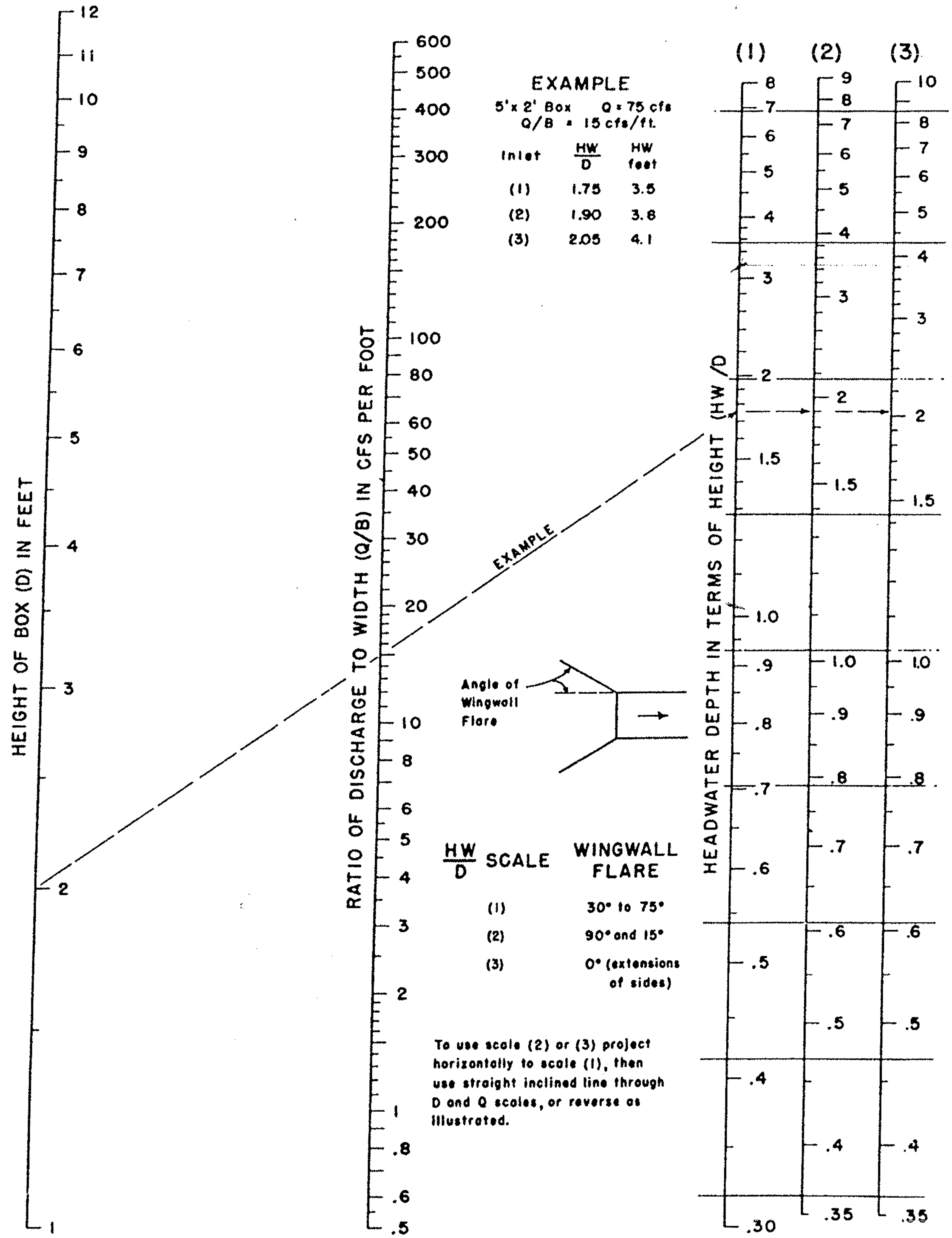


FIGURE 4C-2. Headwater Depth for Circular Pipe Culverts With Beveled Ring Inlet Control

CHART 8



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

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FIGURE 4C-3. Headwater Depth for Box Culverts With Inlet Control

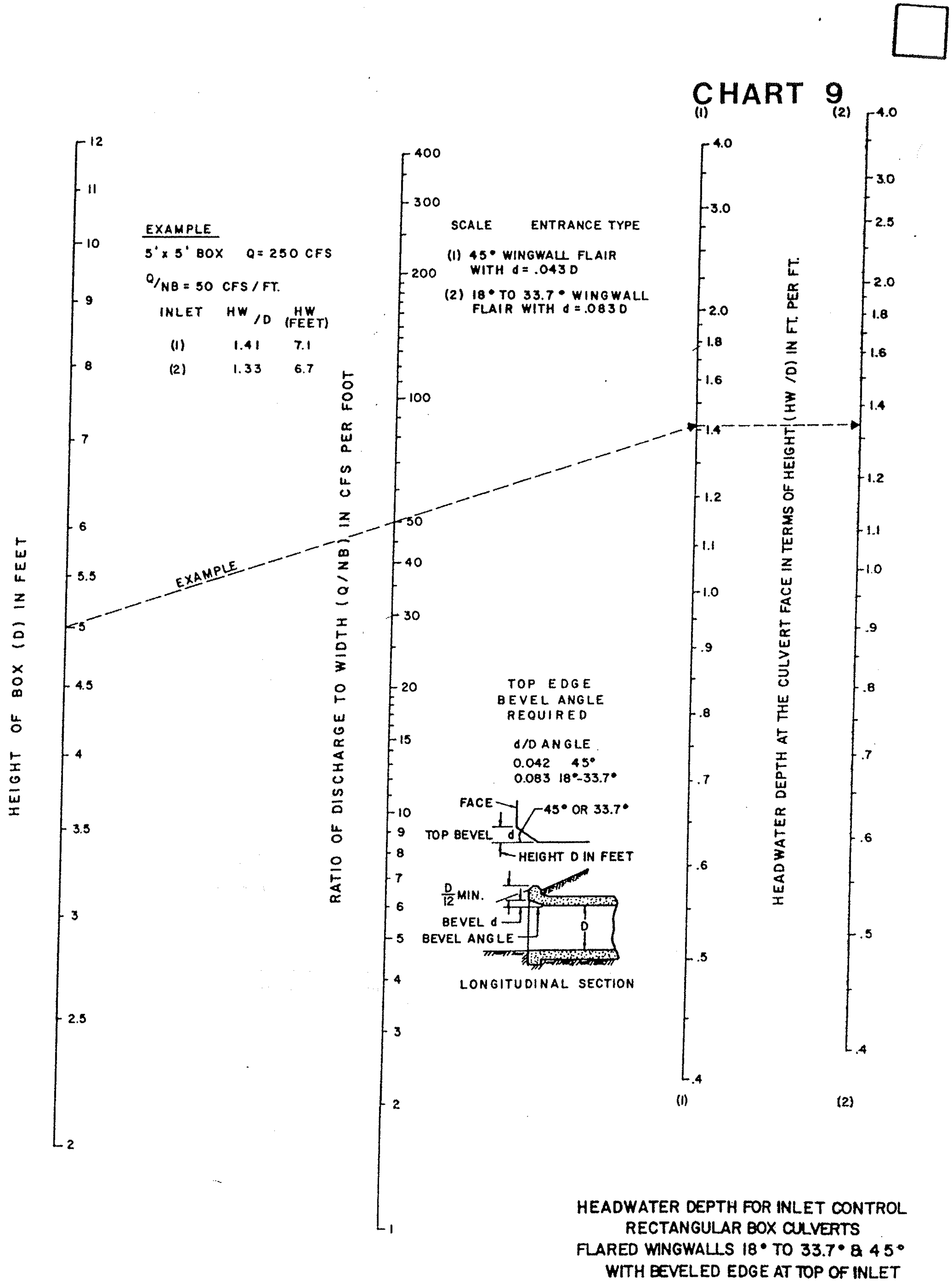


FIGURE 4C-4. Headwater Depth for Inlet Control, Rectangular Box Culverts, Flared Wingwalls 18° to 33.7° & 45° With Beveled Edge at Top of Inlet

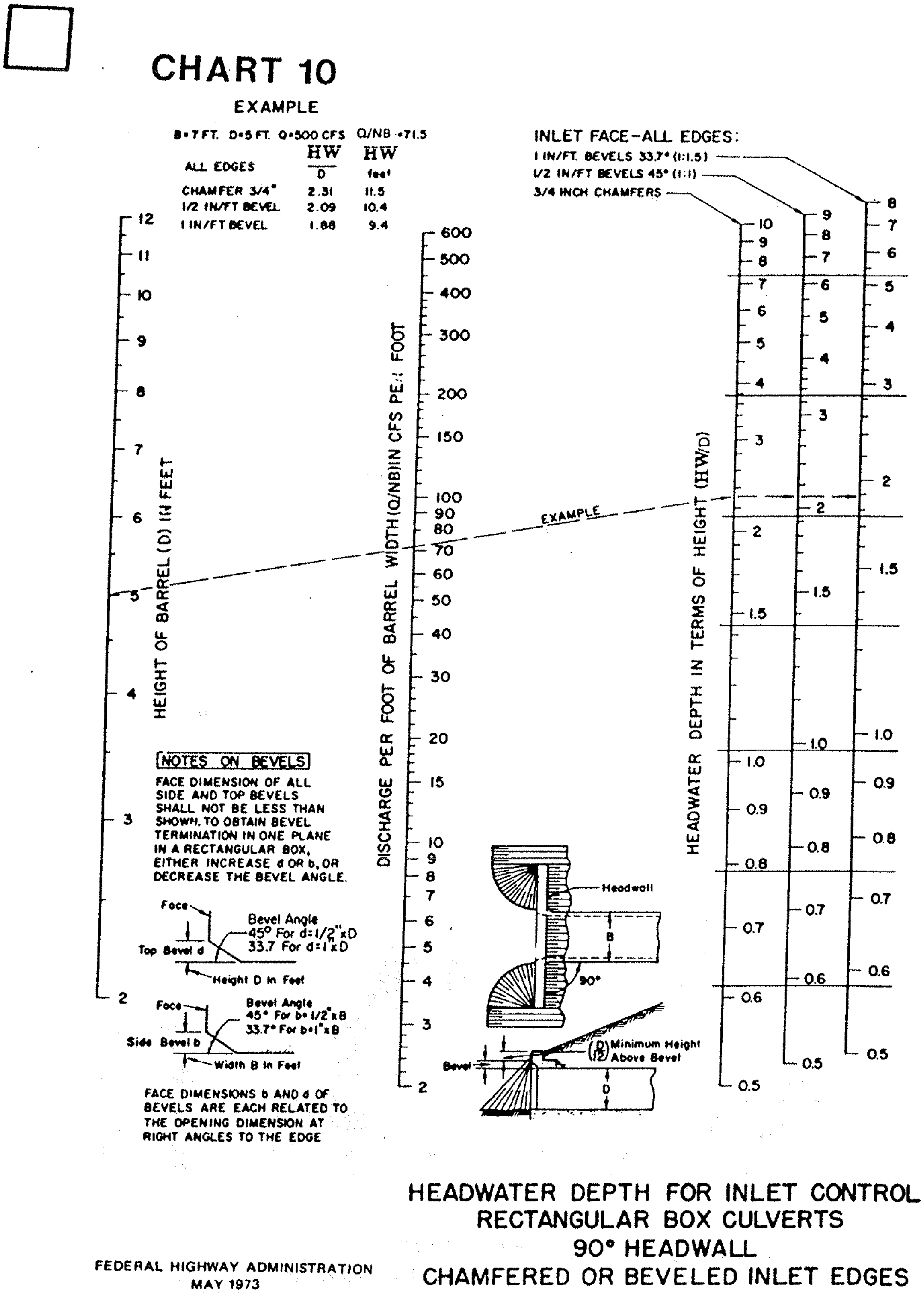


FIGURE 4C-5. Headwater Depth for Inlet Control, Rectangular Box Culverts, 90° Headwall Chamfered or Beveled Inlet Edges

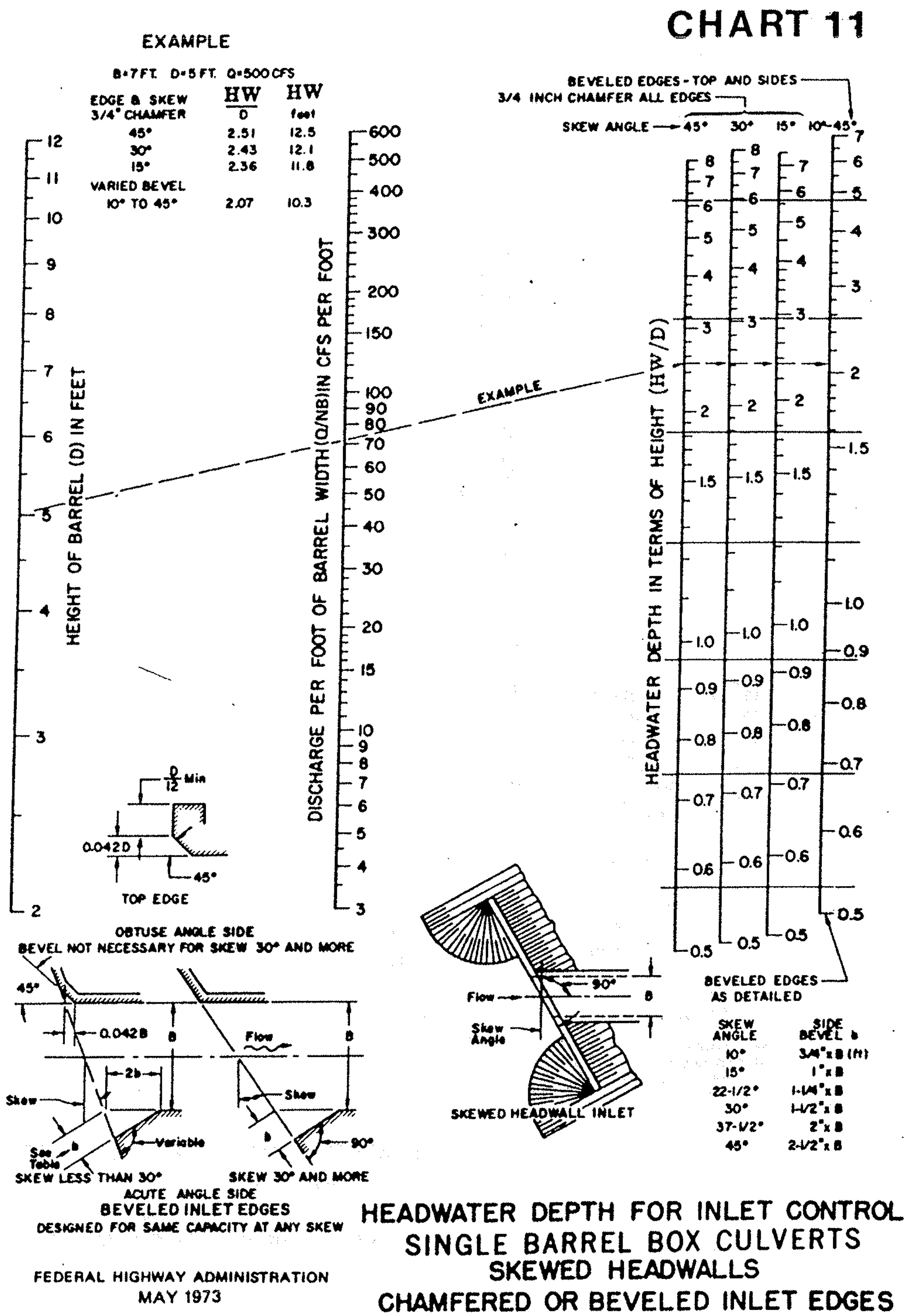


FIGURE 4C-6. Headwater Depth for Inlet Control, Single Barrel Box Culverts, Skewed Headwalls, Chamfered or Beveled Inlet Edges

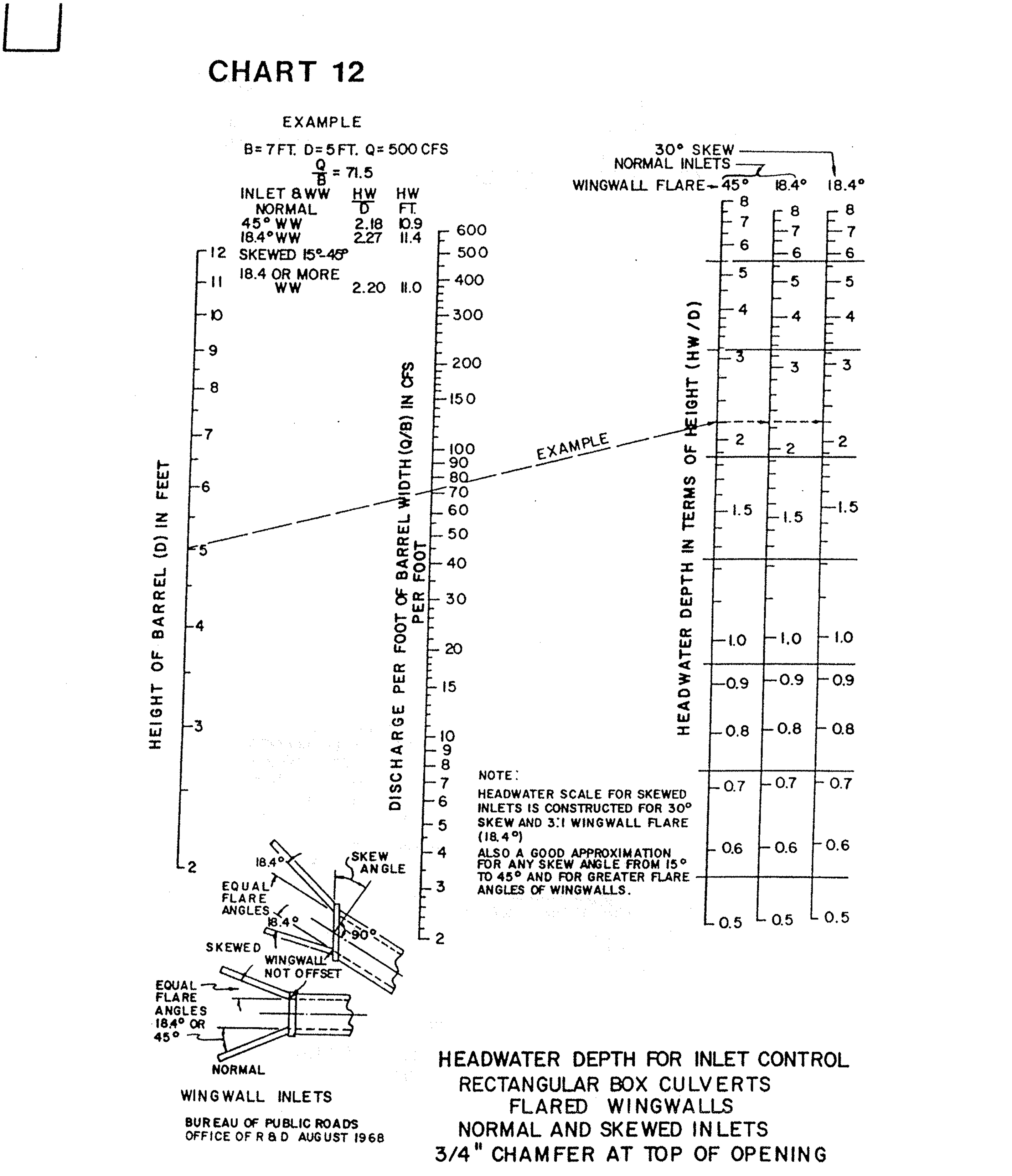


FIGURE 4C-7. Headwater Depth for Inlet Control, Rectangular Box Culverts, Flared Wingwalls, Normal and Skewed Inlets, ¾" Chamfer at Top of Opening

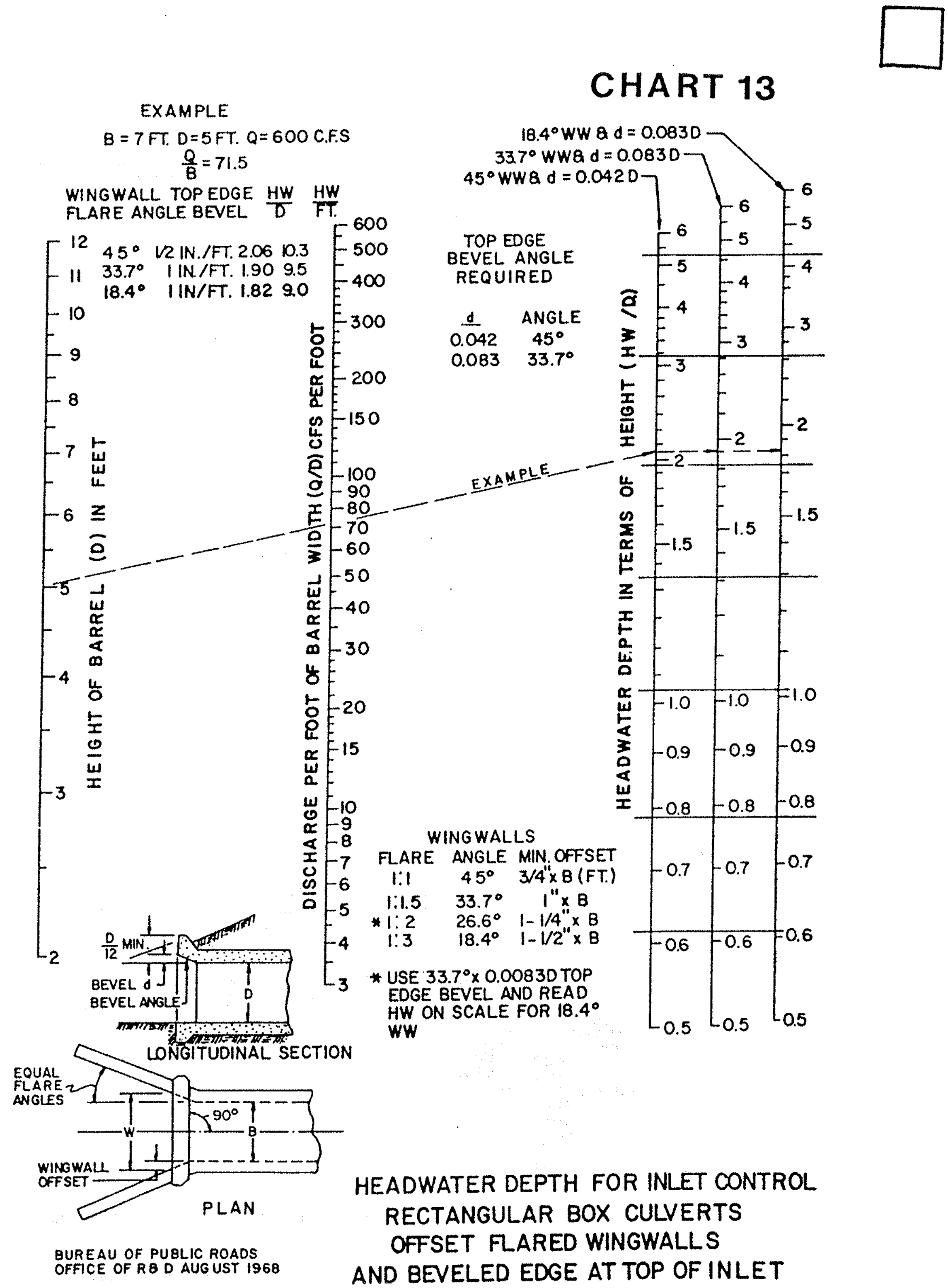
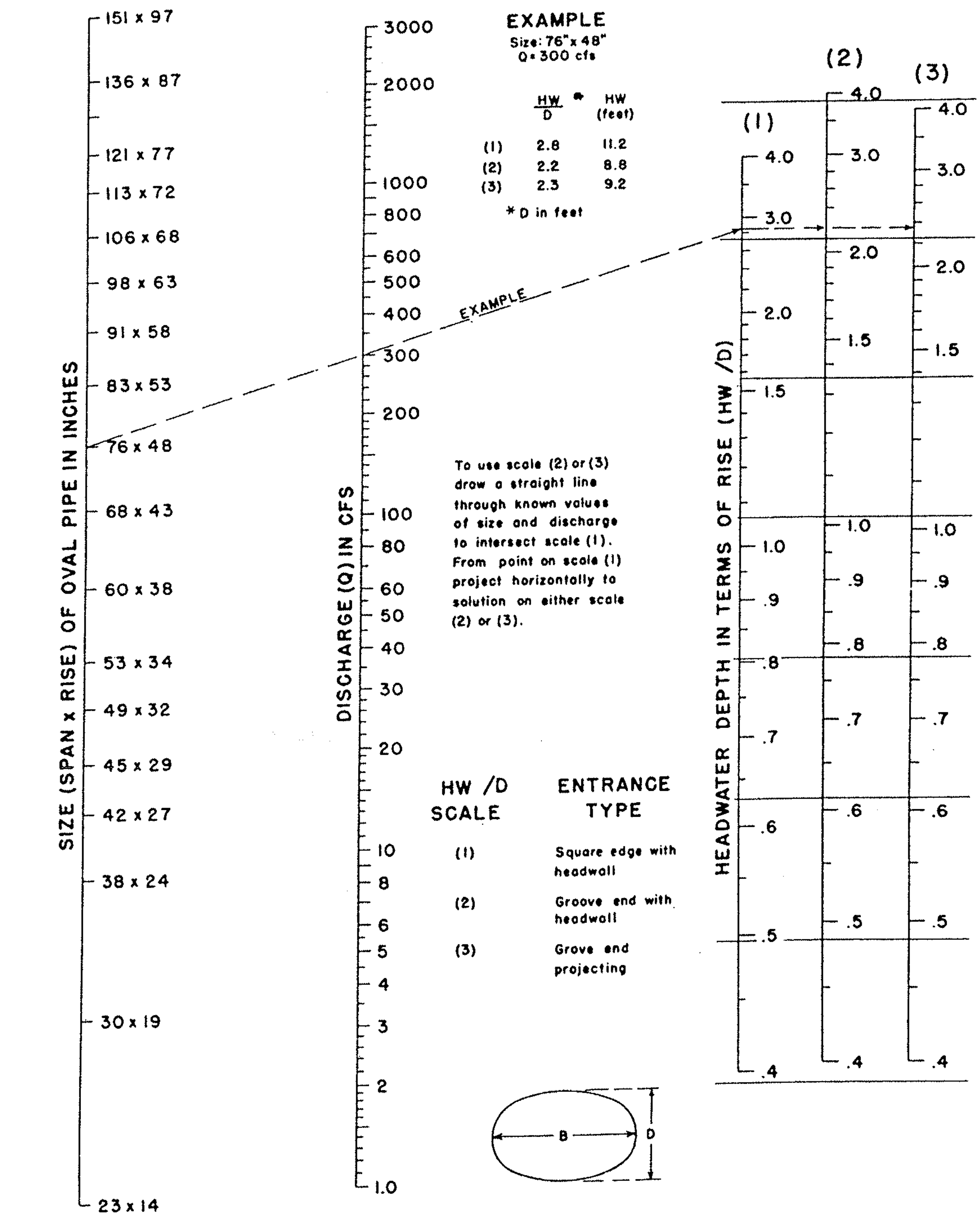


FIGURE 4C-8. Headwater Depth for Inlet Control, Rectangular Box Culverts, Offset Flared Wingwalls and Beveled Edge at Top of Inlet



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

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FIGURE 4C-9. Headwater Depth for Oval Concrete Pipe Culverts, Long Axis Horizontal With Inlet Control

CHART 30

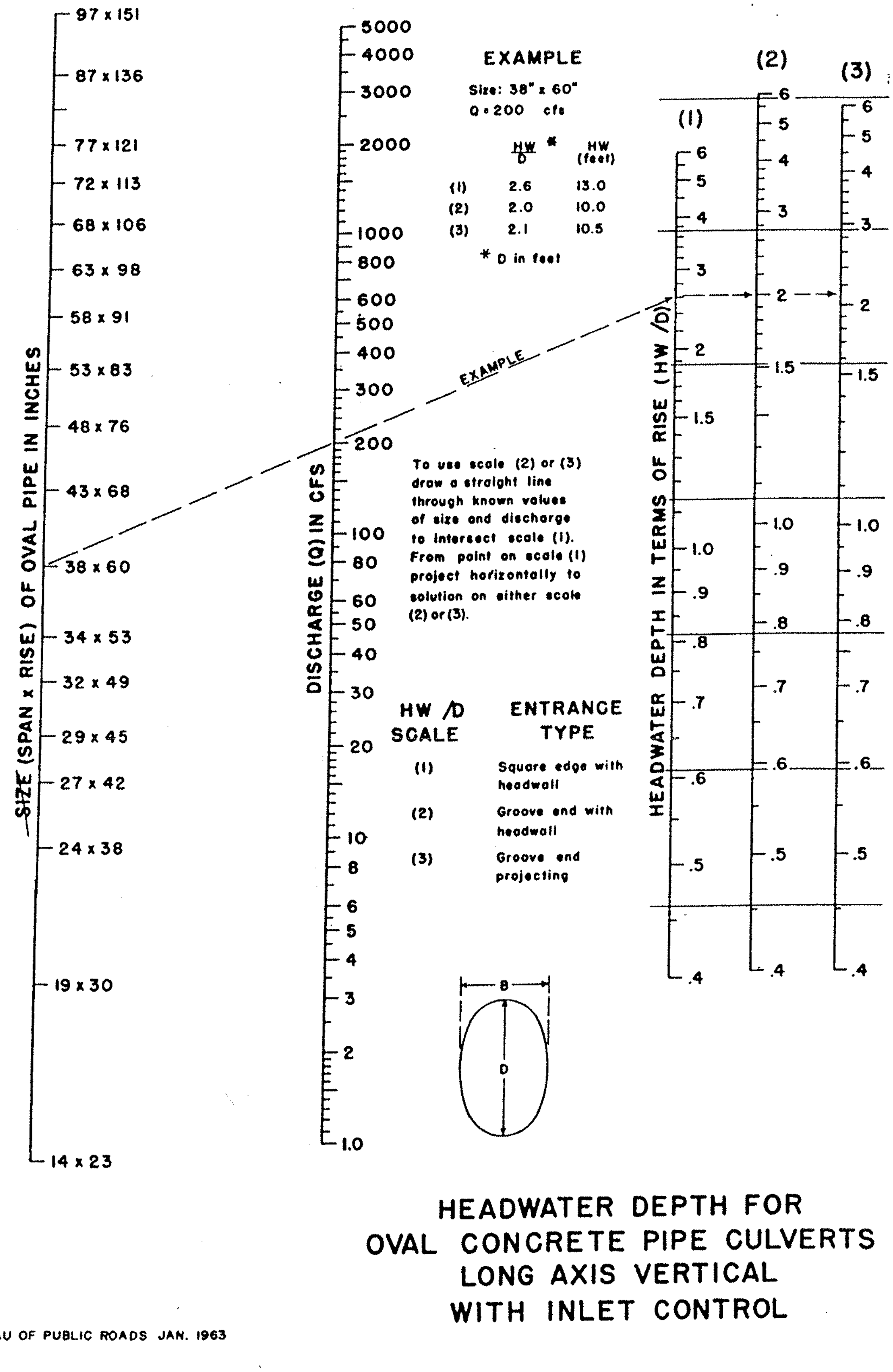


FIGURE 4C-10. Headwater Depth for Oval Concrete Pipe Culverts, Long Axis Vertical With Inlet Control

APPENDIX 4D

HYDRAULIC ELEMENTS CHART
(Source: AHTD)

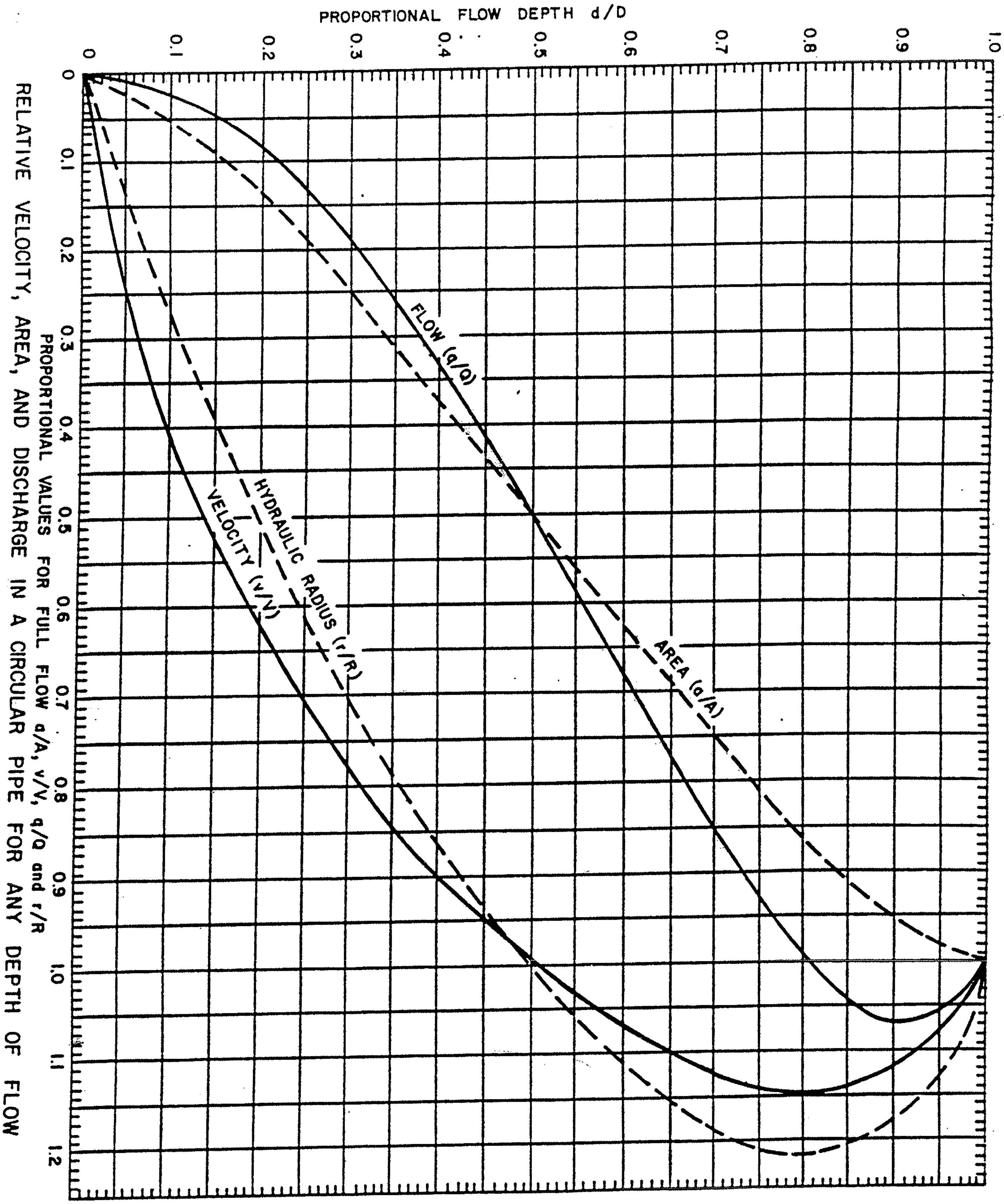


FIGURE 4D-1. Hydraulic Elements Chart.

APPENDIX 4E

EXAMPLE PROBLEM
(Source: AHTD)

FIGUR -1. Example Problem, Culvert Tabulation Sheet

FORM HYD 4-1

PROJECT: *Example No.1*

CULVERT COMPUTATIONS
(SQUARE AND BEVELED EDGES)

DESIGNER: *HYD*

DATE: *11-11-81*

HYDROLOGIC AND CHANNEL INFORMATION

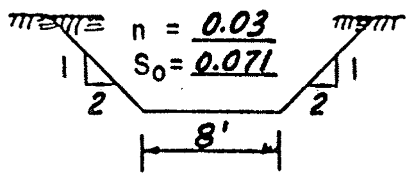
HYDROLOGY

Q_1 *50* = *1000* cfs
Q_2 ____ = ____ cfs

STREAM DATA

Tw_1 = *3.2*
Tw_2 = ____

n = *0.03*
S_0 = *0.071*

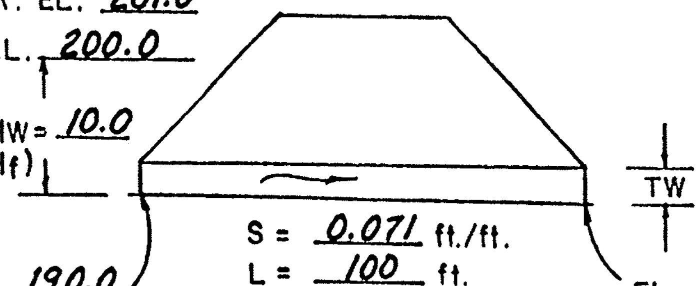


OUTLET CHANNEL
(APPROX. DIMENSIONS)

SKETCH

STATION: *30+00*

FIN. GR. EL. *201.0*
HW_f EL. *200.0*
AHW = *10.0*
(H_f)
H_f EL. *190.0*



S = *0.071* ft./ft.
L = *100* ft.
EL. *182.9*

TRIAL NO.	SIZE	STRUCTURE TYPE & ENTRANCE DESIGN	Q — Q/NB — $\frac{Q}{NBD^{3/2}}$	HEADWATER COMPUTATION — OUTLET CONTROL — (a) K_e	H	(b) d_c	(c) h_o	(d) TW	LS	(e) HW_o	INLET CONT. — $\frac{HW}{D}$ (f)	HW	CONTROL-LING HW	OUTLET VELOCITY ft./sec.	COST	COMMENTS
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	8x6	SQ. EDGE	125	0.4	11.0	>6	6	3.2	7.1	9.9						CLOSE TO AHW - TRY 7'x6'
2	7x6	"	143	0.4	15.0	>6	6	3.2	7.1	13.9						EXCEEDS AHW - CHECK TRIAL 1 FOR BEVELED EDGE
3	8x6	BEVELED	125/8.5	0.2	9.5	>6	6	3.2	7.1	8.4	3.1	18.6	18.6	21		LOWERED HW_o 1.5'. H_f EXCEEDS AHW - TRY SIDE-TAPERE

(a) Entrance loss coefficient, Refer to Table 4-10, page 4-60.
(b) "d_c" cannot exceed D.
(c) h_o = $\frac{D}{2}$ or TW, whichever is larger.
(d) TW = d_n in natural channel, or other downstream control.
(e) HW_o = H + H_o − LS
(f) Use Chart 4-7, page 4-64, for Conventional
Use Chart 4-8, page 4-65 for Beveled E